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VOLUME 91

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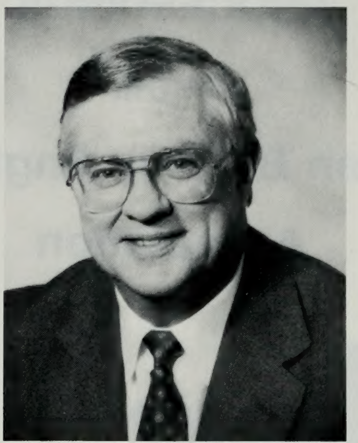

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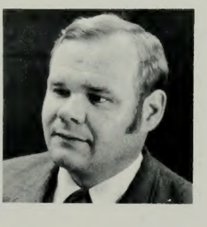

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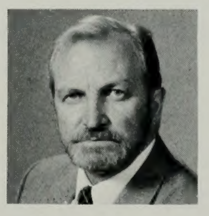

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Front Cover Photo - Railway Track and Radio Telescope antennas at Very Large Array west of Socorro, New Mexico July 1989.

Rear Cover Photo - Westbound CSX train between Germantown and Boyds, Maryland October 1988.

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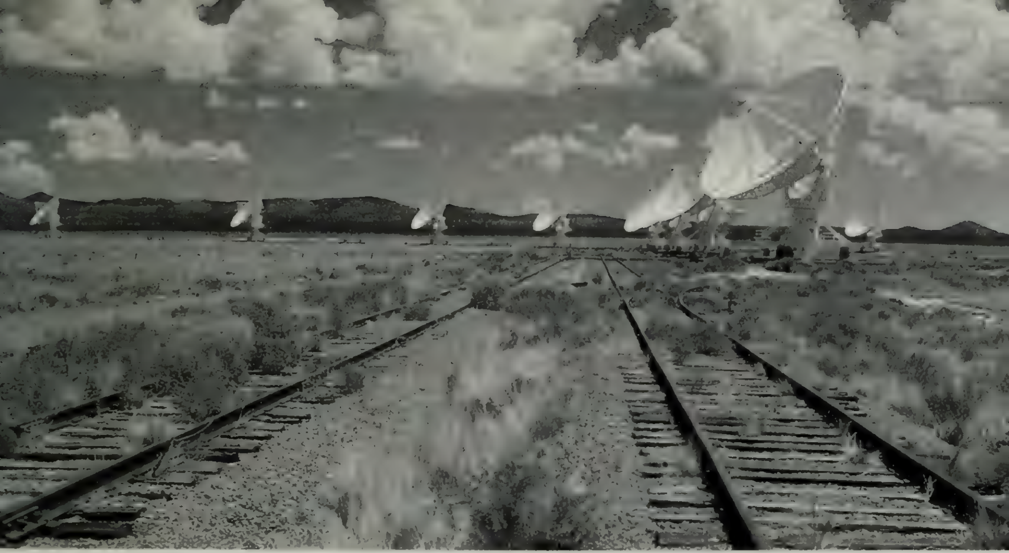
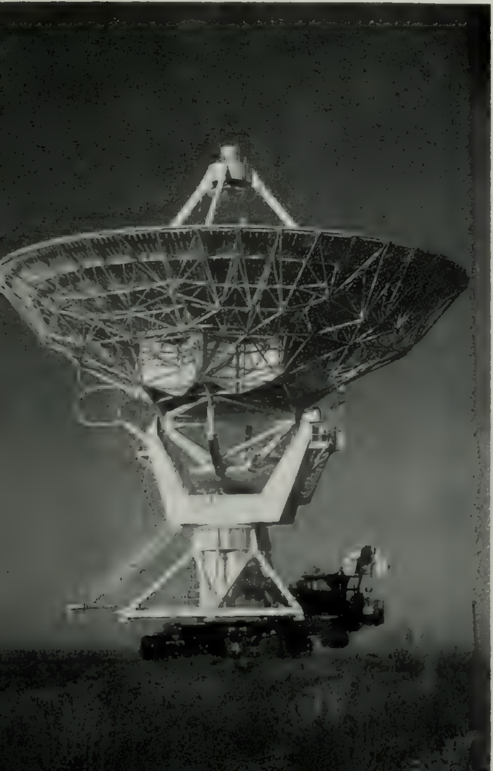


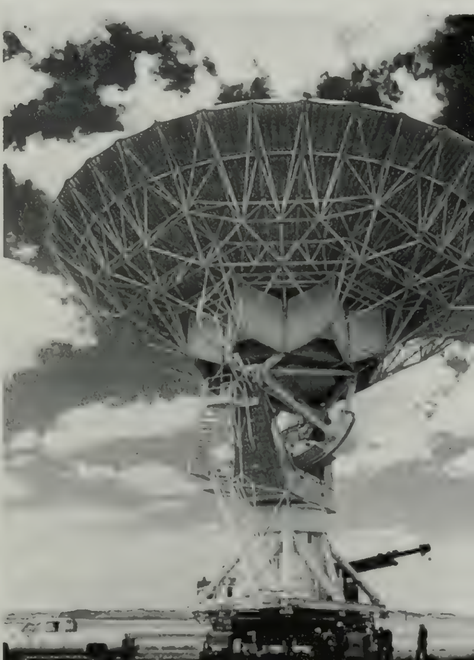
Photo 1—Railway tracks for moving VLA radio antennas (turnout on right track is to M of W facility).

RAILWAY TO THE COSMOS

The technology of conventional railway track is useful when any heavy object has to be moved, and so is sometimes put to uses other than the normal one of carrying railroad trains. One such use is the movement of individual antennas in the Very Large Array (VLA) radio telescope located west of Socorro, New Mexico. The radio telescope is basically about 25 miles in diameter consisting of 27 identical reflector antennas that are moved to different locations on 3 spokes radiating from a center area. Each spoke consists of double track standard gauge railroad, with the two tracks 18 ft. from center

Photo 2, left, shows side view of transporter, while Photo 3 shows transporter at 90 degree junction as hi-rail stands by on main track.





line to center line. The antennas are mounted on platforms on short track segments of double track at right angles to the spokes. Connections between the spokes use conventional turnouts and diamonds.

The antennas are moved by a transporter platform (see photos 2 and 3) which runs on four specialized 3-axle trucks (two on each track). These 3-axle trucks are capable of rotating 90 degrees, so at the 90 degree junctions with the short tracks to the antenna platforms, the transporters with the antenna can be raised on jacks and the 3-axle trucks turned 90 degrees and lowered on to the short perpendicular tracks. The antennas are moved to new configurations about 4 times a year to allow astronomers to observe a wide variety of radio sources (different antenna configurations are best for different types of astronomical objects) and also to move the antennas to and from enclosed shelters where they can be serviced. One of the reasons for selection of a railroad instead of a road was to have the smallest effect on the desert ecology. Hi-rail vehicles create almost no dust such as would be raised by vehicles on maintenance roadways, and as is evident from photo 1, the local vegetation doesn't seem to mind the railroad at all. Because of the low speeds and relatively low axle loads, the vegetation doesn't affect the strength of the track to a significant extent.

The VLA is an astronomical observatory which uses radio waves instead of the normal light waves we see with our eyes. Just as an astronomer at an optical telescope can record photographic images of the object being studied, a scientist using the VLA can make an image using the radio waves that are emitted by that astronomical object. These images can be finer and more precise than the detail in the best photographs taken by the larger optical telescopes and thus the VLA radio telescope is very valuable, especially for studying distant galaxies, some of which are strong emitters of radio waves. The 27 antennas of the VLA are made to behave somewhat as if the whole area of the VLA were a single large telescope. A radio telescope also has the advantage of not being blocked by cloudy weather since radio waves are not substantially reduced by clouds. In addition to its outstanding capabilities as a radio telescope, the VLA is also an extremely powerful radio receiver, which enabled the recent Voyager II

Photo 4. The long, 13-mile-long spokes create the unusual sight of a light-density double track railroad in an unpopulated area.

space craft to transmit a much greater amount of data from the planet Neptune than would otherwise have been possible. The VLA has also been used recently to receive signals reflected off the Saturnian satellite Titan. The variation in reflected signals received apparently indicates that, under its thick clouds, Titan has continents and oceans, although the oceans consist of methane instead of water.

In order to save funds, the trackage for the VLA was constructed entirely of used material installed from 1974 to 1977. The wheel loads are lower than those in normal railway operation and the speeds are generally at 10 mph or less, creating a situation where used material is completely adequate for the loads. However, it has been a mental strain on some of those oriented to our future in space to depend on used railroad material, over 70 years old in some cases. A tie replacement program replacing the worst ties first using a small track gang is now used to maintain the track much as a low density short-line railroad would. Because of the extremely low tonnage and low speed, rail wear and deterioration would appear to not be a problem in the forseeable future. In addition, tie life is helped by the dry conditions in this desert setting and also the deterioration-inhibiting effects of the 7,000 ft. elevation. AREA member Guy Stanzione is in charge of track maintenance in this facility and cares for the 80 plus miles of track involved with a small gang of maintenance people. While it is only 80 miles of track on a New Mexico desert, the railroad at the VLA is a critical link in bringing information from the outermost reaches of the known universe to earth.

Excerpts From Annual Report of A.R.E.A. Committee 1 - Roadway and Ballast

W. S. Stokely, Chairman

Brief status of each Subcommittee Assignment:

Subcommittee 1. Roadbed

Assignment D1-1-86, a report on the criteria justifying the use of under-cutters, was presented in its final draft at the Spring 88 meeting, and voted for publication with minor changes. The Chair expects this shortly.

Assignment D1-2-88 is to update the Manual text on construction of roadbeds. This is a new assignment, and work has not begun account activity on the other assignments. Progress is expected during calendar year 1989 with completion being moved to 1991 from 1990.

Assignment D1-3-88 is on railroad applications of mechanically stabilized walls. Work on this matter has progressed to a final draft and will be available during the coming Fall meeting. Upon the completion of this work, this topic will become the charge of Committee 8, Concrete Structures and Foundations.

Subcommittee 2. Ballast

Assignment C2-1-86 deals with reporting on result of AAR and other ballast research programs. The efforts continue now in the monitoring of the operation of the FAST/HAL Ballast Tests. This assignment is, of course, on-going to the extent that ballast tests are in progress.

Assignment D2-1-88 dealing with tamping procedures and their relation to the ballast. Planning continues with work expected. Our time frame is yet unsettled.

Assignment D2-2-89 dealing with Hot Plant Mixed Asphalt courses in the track substructure is a new assignment. Subcommittee leadership expects to submit a progress schedule at the coming meeting.

Assignment D2-3-89 deals with comparing in-track performance of ballast with laboratory results obtained by the Mill Abrasion Test. Also, an attempt will be made to specify the Mill Abrasion Test.

Assignment D2-4-89 addresses the effect of in-track degradation of ballast particles by weather, and chemicals of the atmosphere and lading. Organization of this task is not complete.

Subcommittee 3. Natural Waterways

Assignment D3-1-86 has been presented. The second assignment, D3-2-88, on the topic of protection of roadbed and bridges from washouts continues. Assignment D3-3-89, a glossary, is new. Subcommittee leadership expects not more than 24 months, including all handling, to publication.

Subcommittee 4. Culverts

This Subcommitee has two assignments, D4-1-88 on the matter of updating the Manual text on installation of pipe culverts, to include jacked installations. Progress is noted with completion expected in 1990. Assignment D4-2-89 on installation of culverts inside existing culverts is new. A 30 month time frame to Manual Revision is expected.

Subcommittee 5. Pipelines

The Subcommittee on Pipelines has two assignments. Assignment D5-1-74 deals with casing pipes larger than 42″ in diameter. Assignment D5-2-81 deals with plastic carrier pipes.

The first Assignment, 15 years old, is coming to a close, it appears. The Subcommittee has developed a Personal Computer based calculating routine which permits the sizing of the casing pipe for the range of diameters currently in use. Checking of the results of these runs is the remaining task.

With D5-1-74 drawing to a close, the remaining assignment, on the use of plastic carrier pipes, will, hopefully, get more attention.

Both assignments are important to the railroad industry, and the effort to update the Manual must continue.

Subcommittee 6. Fences

This subcommittee has two assignments. Assignment D6-1-87, dealing with security fences, has been reported out of Subcommittee. The Committee Chairman has returned the work with technical and graphics quality comments. The expected 1989 completion is, however, delayed.

Assignment D6-1-87 deals with snow fences. The Subcommittee Chairman indicates the current Manual material may be technically in-correct and intends to propose withdrawing the materials pending study.

Subcommittee 8. Tunnels

This Subcommittee has two assignments. Assignment D8-1-87 addresses updating of the Manual materials, and the D8-2-87 is to report on the Rogers Pass Project. The two assignments go hand-in-hand, and are to continue to a completion in 1989 for the report work, and 1990 for the Manual revisions.

Subcommittee 9. Vegetation Control

Subcommittee on Vegetation Control has two assignments, D9-1-88, development of a commentary, on which progress is reported, and D9-2-89, updating its Manual Chapter, a new assignment, which at present is only in the organization stages.

Subcommittee 10. Geosynthetics

Subcommittee 10 on Geosynthetics has completed two assignments. The results of Assignment D10-1-86, dealing with geotextiles in erosion control applications, and Assignment D10-2-86, dealing with drainage applications of geotextiles, can be found in AREA Bulletin 719 as proposed Manual Changes.

The Subcommittee expects to address Assignment D10-3-88, geotextile stabilized walls and slopes, promptly, and its new assignment, D10-1-89, geosynthetics for stabilization of track roadbed, will begin with a topic limited panel discussion during the coming Fall meeting of the Whole Committee.

Excerpts From Annual Report of A.R.E.A. Committee 2—Track Measuring Systems

W. M. Worthington, Chairman

The Committee has six (6) Subcommittees functioning as follows:

Subcommittee 1—Investigate characteristics of corrugation.

Assigned: March 1, 1985
Estimated Completion: June 1990

The committee presented a paper of its initial findings at the 1987 spring technical conference. Work is now proceeding on a second phase (with a new chairman) using Jackson Jordan data collected on the CP and correlating it with CP Track Geometry Car and their corrugation reference device. Our initial findings were based on small stretches with data collected by hand. This phase will expand the amount of data available for analysis and more accurately relate it to track alignment and track stiffness if possible.

Subcommittee 2—Establish, through an internal survey, the need for track survey data of different types and the requirement of each data element to be updated.

Assigned: March 1, 1985
Estimated Completion: June 1990

The subcommittee is continuing its discussions on collection and storage of track geometry data. The results of the questionnaire circulated earlier have been tabulated and the information is being used in our discussions. We are working on developing guidelines for what information should be included in a track geometry database. This includes types of data, number of test runs (i.e. for degradation studies), sample intervals, etc.

In the coming year, we plan to continue working on developing these guidelines. We are also trying to improve attendance at our subcommittee meetings, possibly by arranging some that are separate from the full Committee 2 meeting.

Subcommittee 3—Evaluate the need and potential use by railways of information on track modulus, subgrade moisture and lateral tie strength, each of which can be evaluated with current technology.

Assigned: March 1, 1985
Estimated Completion: June 1990

This assignment was initiated in 1985 and has been worked on by the subcommittee under the leadership of three chairmen to date. In order to evaluate the need by Railways of the three aspects of track and structure mentioned in the assignment as well as other possible areas of interest, two questionnaires were prepared and sent out to various railways, suppliers/consultants and governmental agencies. The questions developed for the questionnaires were done so using twelve (12) possible areas of interest that may be needed by a user railway and the availability of devices or technologies which may supply a solution to the need. Each possible device was explained and the respondents were asked to comment on the need, or viability of such a product, device or measurement. The results were then ranked in order of interest shown.

The results of this ranking were then presented to the committee and interested parties as part of a paper prepared for the March 16, 1988—"Symposium on Automated Track Measurement" in Chicago. This brings the assignment to approximately 90% completion having only to prepare a final document of our findings and present it to the general committee. This is expected to be ready by fall of 1989. The three most needed areas of investigation are:

1. Rail wear measurement
2. Lateral rail stress analysis
3. Rail flaw detection

We not only recommend the continuation of the present assignment to completion but we are also going to delve into a new assignment to investigate the three areas of Railway needs arising out of these questionnaires.

Subcommittee 4—Propose recommended practice for geometry cars.

Assigned: March 1, 1985
Estimated Completion: June 1990

To complete its assignment the Subcommittee compiled an inventory of track geometry cars in use in the Free World. This inventory contained specifics on this measuring equipment in use on these cars, the type of car and its capabilities. Advertisement of this inventory was made in RT&S and enjoyed a strong response for copies. Current plans call for updating this survey.

In compiling the survey, a need was seen to redefine certain track terms to bridge terminology differences amongst suppliers and users of track geometry cars and equipment. A letter balloting procedure is now underway to arrive at a consensus on these definitions. We hope to publish the new definitions in the 1990 bulletin.

Two members of the subcommittee, R. N. Harbuck and R. L. Engle of the Santa Fe made presentations at the symposium of Automated Track Measurement in March 1988.

Subcommittee 5—Survey Data Processing Techniques and equipment used on different railways' Track Geometry Vehicles.

Assigned: May 1, 1985
Original Estimated Completion: June 1989
Current Estimated Completion date: June 1990
Percent Completion: 40%

Subcommittee efforts to date have been directed at developing a general definition of Track Quality Indices which are at the heart of nearly every railway's track geometry data processing techniques. A definition has been developed and is now being submitted to the committee by ballot for inclusion in the AREA manual.

Based on this general definition a survey questionnaire has been developed and will soon be distributed to committee members. The results of this survey, and appropriate follow up, will be used to prepare a report to the committee documenting current data processing techniques in use.

Progress has been discontinuous for this subcommittee largely due to the instability of the subcommittee membership. Members have not regularly attended committee meetings, and some have been drawn to meet with other subcommittees. Efforts have recently been made to form a 3 to 4 man "core" membership of those which are committed to accomplishing the subcommittee objectives.

Subcommittee 6—Rail flaw detection.

D6-1-86. Develop a recommended practice describing minimum acceptable flaw detection system capabilities.

D6-2-86. Develop a recommended practice using rail specimens to stimulate defects of known size and orientation to demonstrate minimum rail flaw detection system capabilities.

C6-1-86. Interface with organizations involved in the development of rail inspection technologies.

C6-2-86. Inform the membership on the state-of-the-art in rail flaw detection.

Assigned: June 1, 1986
Estimated Completion: June 1990

A draft specification of minimum acceptable performance for rail flaw detection systems was presented at the 1989 meeting at the Committee on Rail Testing and was subsequently circulated for comments from the CORT membership.

Excerpts From Annual Report of A.R.E.A. Committee 3 - Ties and Wood Preservation

C. L. Hardy, Chairman

Status of Subcommittee Assignments:

Subcommittee A—Recommendation for further study and research.

No recommendation for further study at this time, however in addition to watching the development of imported timber, we will review the computer models for use in predicting tie performance.

Subcommittee B—Revision of Manual.

Some manual revisions will be forthcoming due to changes in the AWPA specifications. As soon as these specs are reviewed a manual revision will be requested since AREA and AWPA specifications are the same.

Subcommittee No. 1—Cross and Switch Ties.

C1-2-63. Keep up to date specifications for cross and switch ties.

The specifications are updated as needed, however there has been no need for changes in the past few years.

C1-1-62. Investigate possible revision of cross tie design and/or spacings.

Again there has been little need for changes in this area, however there is activity in wood tie fasteners which could impact tie design and spacing sometime in the future.

D1-1-89. Wood tie disposal alternatives.

We are gathering information on this important subject such as co-generation, controlled burning and use in reconstituted ties.

Subcommittee No. 2—Wood preservatives and preservative treatment of forest products.

C2-1-63. Keep up to date specifications for preservatives and update research for new preservatives.

As mentioned above, we are reviewing the revised AWPA specifications and will recommend manual changes accordingly. We continue to observe possible environmental problems in this very sensitive area.

C2-2-63. Keep up to date specifications for seasoning, conditioning and treatment.

As with preservatives, this assignment could also be an environmental issue. We are studying the possible impact of proposed legislation.

C2-3-63. Advisability of preparing specifications to cover care and handling of forest products before and after treatment.

This section is similar to the other parts of the Subcommittee assignment in that there has not been much need for change during the past few years, but possible environmental issues may soon necessitate revisions.

Subcommittee No. 3—Service records of forest products.

C3-1-63. Inform AREA about annual tie renewal statistics as furnished by the Economics and Finance Department, AAR.

Information concerning annual tie renewal is furnished each year.

C3-2-63. Keep up to date with service test records of forest products used in railroad construction and maintenance.

A list of test in place on railroads has been compiled and is updated as conditions change. These test zones are often candidates for field trip observations.

Subcommittee No. 4—Collaborate with AAR Research Department and other organizations in research and other matters of mutual interest.

C4-1-76. Substitute for solid-sawn wood ties.

While the demand for laminated ties has become less important due to an adequate supply of solid-sawn ties, the interest in reconstituted ties has increased because of tie disposal problems and predicted increased durability. We expect some conclusions in this area soon.

C4-3-63. Splitting of ties and anti-splitting devices.

Anti-splitting devices are becoming common in both cross and switch ties. We are looking to bringing this assignment to a conclusion within one year.

C4-3-66. Wood deterioration in the presence of metal.

We are studying the results of research into this subject and continue to observe the problem.

C4-4-82. Monitor progress of work of each subcommittee and assist with an appropriate exchange of technology between AREA and AAR.

This assignment is progressing well as was witnessed by the Committee/AAR inspection at Pueblo.

While most of the subcommittee assignments are on-going, as mentioned above a few are about to be concluded. There are none to be dropped or modified at this time.

Excerpts From Annual Report of A.R.E.A. Committee 4 - Rail

A. W. Worth, Chairman

Brief status of each Subcommittee Assignment:

Subcommittee A

Subcommittee A is assigned with the recommendations for further study and research and is a continuing committee.

This committee has recommended that a look be taken at increasing the minimum yield of standard AREA rail to 300 BHN. Some preliminary correspondence is under way with the rail manufacturers and it is expected a recommendation will be made to the Board of Direction that the subject be assigned to Subcommittee C2-1-55.

Subcommittee B—Revision of Manual

A continuing committee. All the requested manual revisions have been assigned to the appropriate subcommittees.

Subcommittee 1

C1-1-74. Collaborate with technical representatives of rail suppliers, welding contractors, suppliers of field welding, rail grinding and rail testing contractors on matters of mutual interest.

A continuing committee, necessary to maintain liaison between railroad members and suppliers and keep all concerned informed of ongoing improvements and changes in technology.

D1-3-83. Ad Hoc Committee on Rail Profile Grinding

Assigned in 1983. Final report of this subcommittee was published in the AREA Bulletin for December 1988, pp. 471-487, and it was presented at the March 1989 Technical Conference in Chicago by Dr. A. M. Zarembski. We have recommended that the subcommittee be discontinued. If in future further definite objectives for study and report are evident a new subcommittee can be set up.

D1-2-81. Weld Qualification Testing

Assigned approximately 1981. Work has been proceeding at the AAR Research Center in Chicago. The report as regards carbon steel rails is now complete in draft form except for some photos, and should be complete this year. As it was problems with various types of alloy steel rails that originally provoked the study, and it is not clear that alloy steel rails are addressed, it may appropriate to request a new "D" series subcommittee be set up to report on that subject.

D1-1-81. High Strength Rail Welding

Assigned approximately 1981 (or earlier). When started, this subcommittee concentrated on welding techniques for chrome alloy rails, but the chairman resigned before any report was submitted and the committee became inactive. Committee was re-started 1985 and since then has concentrated mostly on welding heat treated rails. Purpose of subcommittee is to develop and standardize techniques for making welds in high strength rail that will not be subject to brittleness, batter or spalling. The chairman has been trying to write his report since the beginning of 1989, but late information and changes keep coming in at irregular intervals and extent of information input is extremely variable. He still hopes to have the report completed by the end of 1989 but if slippage continues a date in 1990 may be inevitable. A very worthwhile subject that should be carried through to a conclusion even if a little more time is needed.

Subcommittee 2

C2-1-55. Collaborate with technical representatives of rail and joint bar manufacturers in research and other matters of mutual interest.

A continuing committee, necessary to maintain liaison between railroad members and rail and joint bar manufacturers and keep all concerned informed of ongoing improvements and changes in technology. Research work is ongoing on ultrasonic testing, stamping, bar coding, straightness, acceptance gauges and tolerances.

D2-1-88. Web Cracking of Rail from Roller Straightening

This is the AREA designation of an ad hoc subcommittee set up between the AAR, the North American steel mills and a number of U.S. and Canadian railroads in January 1988 on an emergency basis, to investigate causes of split webs in newly received rails from a number of mills. An interim report was given to the Board of Direction in November 1988 and a presentation of that data was made by Dr. R. K. Steele at the March 1989 technical conference in Chicago.

A systematic investigation requires that effects on residual stresses be checked at all mills, on rails of different types and straightnesses, and a logic matrix set up. Up to now a major difficulty has been the great difficulty and expense of getting reliable residual stress measurements. An optimistic schedule of work completion (which depends on such measurements going well) calls for a report in 1990. The Board of Direction has agreed that anything that will assure soundness of rail in track under load is desirable and it is recommended this study by continued.

D2-2-88. Tolerance and Method of Measuring Rail Head Radius

Assigned 1988. Many railroads have commented that though the designs for rail head radii on the AREA plans are based on very precise assumptions as to wheel-rail contact the head radii on new as-rolled rails are likely to be sufficiently different from design to alter the contact patterns substantially. Differences in head radii affect contact stresses in wheels and rails and affect wheel tracking.

The chairman of this subcommittee has been working with the chairman of D5-2-88 to try to formulate a joint approach, because without the data from D5-2-88 he does not know where to start. As a result, the subcommittee has not yet had its first meeting. Completion should coincide with that of D5-2-88, circa 1992.

Study should be continued. There was some discussion of combining this subcommittee with D5-2-88, but D5-2-88 is a pure railroad committee of a scientific and technical nature and D2-2-88 is a commercial liaison committee with suppliers. In the end it was decided to keep them separate, to avoid danger of a scientific report being held up by commercial concerns.

Subcommittee 3

C3-1-76. Rail Statistics

Assigned 1976. This committee reports in a standardized format each year the rail shipped to North American railroads from North American and non-North American producing mills, by weight and section. To ensure confidentiality, all data is sent to an independent accounting firm to be consolidated and information as to origin and destination is deleted. Report for 1987 was sent to AREA headquarters in July 1989. A continuing committee; a worthwhile subject that should be continued.

Subcommittee 4

C4-1-73. Update data on methods and equipment for making welding repairs on rail and turnouts including thermite welding.

A continuing committee. This committee maintains liaison with the AWS, which is active in the field of trackwork welding, and with AAR in connection with research at the FAST loop.

The AWS recommended procedure for track welding is expected to go out of Subcommittee to the AWS Technical Activities Committee about September 1990. Also, the AWS will sponsor a welding symposium (including track welding) probably in Chicago in 1992.

No report is available from the FAST loop yet due to insufficient tonnage being accumulated at the time of report. A useful and necessary committee that should be continued.

Subcommittee 5

C5-1-78. Rail Specifications, Research and Development.

This is a very important continuing committee. It has just completed publication of the drawing of the 133 RE section in the manual (page 4-1-5.1) and a reorganization of the material on welding. A further reorganization in the near future is expected to be a complete revision and a reissue of all drawings of both rail and splice bars. Subcommittee 5 is vital to the functioning of the Committee and it is recommended the subject be continued.

Following resignation of the previous chairman, a new chairman was appointed for this subcommittee at the October 1988 meeting in Pueblo and things are going well.

D5-1-84. Recomputation of Physical Properties of Rail Sections.

Physical properties of rail sections now published in the Manual were computed by hand many years ago and when they are checked by modern mathematical computer techniques they are found to be inexact. Also, certain important properties such as lateral moment of inertia are not given at all. Re-publication of the drawings was initiated in 1984 but could not be carried through till the subcommittee decided what exact density should be used for uniform computation of the rail weight.

At the May 1989 meeting the subcommittee decided to use 7.84 gm/cc as a nominal density. Redrafting can now go forward and it is hoped drawings will be ready for ballot and publication in 1990. A desirable task that should be carried through to completion.

D5-2-88. Wheel Rail Contact Stresses.

This was the pro tem designation of an ad hoc subcommittee set up early in 1988 between the AAR Engineering and Mechanical Divisions and a number of major North American railroads to study (a) what recommendation should be made for a new standard AAR freight wheel profile and (b) what changes to standard AREA new rail head profiles and rail profile grinding techniques might be necessary to produce optimum rail life with such new profiles.

The subcommittee chairman has pointed out that the subcommittee might more properly be called "Wheel/Rail Profiles" and formal request has been made that such be authorized.

This is a co-operative committee being pursued primarily through the AAR and is one that should be continued. Due to the complex nature of the subject, report is not forseen before 1992.

Subcommittee 6

C6-1-62. Joint bars, design specifications, service tests, including insulated joints and compromise joints.

A continuing committee. Currently this committee is developing specifications for the fabrication of bonded insulated rail joints, an assignment that is going ahead slowly and is not expected to be complete till 1991. It also has undertaken a complete review of all the current drawings for splice bars now in the manual. The new drawings are complete and if approval is given by AREA management they will be put out for letter ballot for adoption.

Another essential and very productive committee that should be continued.

Subcommittee 7

C7-1-68. Effect of Heavy Wheel Loads on Rail.

A continuing committee. This committee has an ongoing study related to the effects of heavier cars on the fatigue and wear life of rail steel, and such allied phenomena as corrugation, rail batter and problems at joints. It handles the liaison with AAR on rail life prediction computer models, and on results from the present HAL test at the AAR transportation test facility. This is an important committee and should be continued.

Subcommittee 8

C8-1-88. Management of Rail Test Resources.

A continuing committee, which until 1988 was entitled "Recommendations for Interval of Non-destructive Testing for Internal Defects of Rail and Track."

Some initial liaison has been done between AREA and TDC, and some TDC charts presented for discussion, but ideas so far are only in initial stages. The Chairman has been forced to give up the chair (June 1989) due to pressure of other commitments and as of time of writing the subcommittee has no chairman. The matter of a replacement will be put up before the Committee at the meeting November 1.

This is an important committee, but one for which it has been difficult to get really qualified people who would devote time to it. It should be continued and encouraged.

Excerpts From Annual Report of A.R.E.A. Committee 5 - Track

W. B. Dwinnell III, Chairman

Report on Special (D) and Continuing (C) Assignments

Assigned Number	Description of Assignment	Date Assigned/ Undertaken	Estimated % Completion	Estimated Completion Date	Remarks
B-2	Anchor application and maintenance		90%	5/90	Letter ballot submitted
B-3	Remanufactured rail anchors	3/89	30%	9/90	Progress being made
C2-1-79	Track tools	2/79	100%	Ongoing	Letter ballot on aluminum tongs
D4-1-84	Elastic rail fasteners-wood ties	4/86	60%	9/90	
D4-2-84	Hold down fastening-wood ties	4/88	20%	5/91	
D4-3-85	Tie plates-wood ties	4/85	90%	5/91	
D5-1-77	Guard rails	1978	100%	Complete	
D5-3-80	Fastening Agents	1980	100%	Complete	
D5-2-79	Gage plates	1980	70%	9/90	
D5-4-81	Explosion Hardening of Castings	1981	90%	5/90	
D5-5-81	Maint. Parameters	1981	30%	5/91	
D5-6-88	Direct Fixation Fasteners	1988	30%	5/90	
D5-7-87	Frog Castings	5/88	30%	9/90	
D5-8-89	Turnout Design	5/89	NEW	9/91	
D7-1-84	Rail Lubrication	9/86	30%	5/91	Questionnaire submitted
D8-1-84	Ballast Shoulder Width	1/84	100%	Complete	Recommendation forwarded to Committee 1
D8-2-84	Underbalance	1/84			To be reworded
D8-3-85	Lack of superelevation	1985			To be reworded

Excerpts From Annual Report of A.R.E.A. Committee 6 - Buildings

B. R. Schwab, Chairman

Subcommittee Assignments:

D-1-80. Design criteria for CTC dispatching facilities.

A preliminary report was reviewed by the members in attendance. A newly appointed Chairman will take report into review, solicit suggestions from members, and submit a revised report at the Toronto meeting.

D-3-87. Design criteria for wheel and bearing shops.

A final report will be submitted for review at the Toronto meeting.

C-3-84. Design criteria for railway office buildings.

Copies of the revised report were submitted and reviewed by those in attendance.

C-1-73. Design criteria for Car Shops.

Current issues of the report were distributed along with a survey sheet on Spot Car Repair Shop. Comments were requested and will be discussed in Toronto.

D-4-89. Design criteria for Railway Material Management Facilities.

Was approved by the Board and Mr. G. Lavoie will chair this committee. It was noted that response and input was needed on the Subcommittee report D-5 Centralized MW Repair Facility to update to current standards. It was agreed not to add additional assignments at this time.

Excerpts From Annual Report of Committee 7 - Timber Structures

W. Benton III, Chairman

Subcommittee Status:

Assignment A: Recommendations for further study and research.

Timber tie deck specifications being discussed for new subject at the suggestion of the AREA Board.

Assignment B: Revision of Manual.

Working on deleting references to lumber grading rule agencies which are no longer active, revising stress values used in Table 2.7.1 to reflect those currently used by the grading agencies and incorporate the current specs. developed by NELMA for Oak and Maple into Table 2.7.1. Also revising protection for pile cutoffs.

Subcommittee C-1-53: Specifications for Design on Wood Bridges and Trestles

Working on Design of Wood Railway Bridges 1975. Seven items now ready for vote by whole committee then tables for E-80 design to be added. Take about 1-1/2 years to finish. Subcommittee Chairman could use more help. He is carrying the bulk of the work.

Subcommittee D-3-82: Effect of dapping and overhang on stresses in bridge ties.

All work done in connection with this subject except for fine tuning the summary. Waiting for subcommittee chairman to submit to me. Report will be published as information only in AREA Bulletin.

Subcommittee D-4-82: Effect of unit trains on timber trestle components.

Two Chairmen have been in charge of this committee since 1982. Present Chairman not able to devote the time to get this subject going. I believe committee should vote to drop due to 7 yrs. old and nothing really accomplished.

Subcommittee D-5-89: Methods to upgrade existing timber bridges.

Committee just getting started on the direction they will be taking. This report will show the measures a railroad can take without the added costs of building a replacement bridge.

Exerpts From Annual Report of A.R.E.A. Committee 8 - Concrete Structures and Foundations

D. J. Lewis, Chairman

Subcommittee Activities:

Subcommittee A. Recommendations for Further Study and Research

In cooperation with the other two structural committees, AAR, NSF and the University of Illinois, we are participating in the steering committee which is progressing with railroad bridge research. The future direction of this research is to determine the static, dynamic and impact loads that bridges must carry under current loading situations.

Subcommittee B. Revision of Manual

This subcommittee is actively reviewing Chapter 8 to make the presentation consistent with the "official" AREA format. All ballot material will be reviewed by this subcommittee prior to the full committee vote.

Subcommittee 1. Design of Concrete Structures

D1-1-84. Develop specifications for precast and cast-in-place concrete segmental bridges.

Work on the segmental bridge specifications was suspended until the Post-Tensioning Institute's recommendations are available. This information is now available and we are progressing with a recommended practice.

D1-5-87. Investigate applicable impact factors.

Applicable impact factors are a continuing concern. Mr. S. Skaberna's paper entitled "A Review of Studies of Impact on Concrete Railway Bridges," has been published as information. In addition, a questionnaire was sent to Chief Engineers or Vice Presidents of Engineering of the Class 1 Railroads asking for their current design practice. The results were presented to the subcommittee but there has not been a decision regarding recommended changes.

D1-8-88. Review and rewrite Part 2, Reinforced Concrete Design.

This part is in the process of being reviewed and rewritten. The rewritten part was not ready for subcommittee letter ballot at our May meeting but should be ready by our February 1990 meeting.

D1-9-88. Review and rewrite Part 17, Prestressed Concrete Design.

This part is in the process of being reviewed. The rewritten part will follow the D1-8-89 subcommittee letter balloting at May 1990 meeting.

Subcommittee 2. Foundations and Earth Pressures

D2-1-88. Rewrite Part 20, Flexible Sheet Pile Bulkheads.

This part is in the process of being reviewed. The rewritten part was not ready for subcommittee letter ballot at our May meeting but should be ready by our May 1990 meeting.

Subcommittee 3. Durability of Concrete

C3-1-88. Monitor current waterproofing practices.

This is a continuous assignment and just had a few changes incorporated in the 1988 Manual. No other changes are presently anticipated.

D3-1-88. Investigate and report on current methods of protecting concrete.

An informal questionnaire was sent to the committee members and the results are being tabulated. The results along with information from ACI 318 will hopefully develop recommended changes for the Manual.

D3-1-89. Guidelines for field bending of partially embedded reinforcing bars.

This is a new assignment and the subcommittee is beginning to gather information.

D3-2-89. Review and rewrite Part 1, Materials, Tests and Construction Requirements.

This is a new assignment and the subcommittee is beginning to gather information. Assignment D3-1-88 may supply information which should be incorporated in this assignment.

Subcommittee 4. Repair, Restoration and Strengthening of Concrete Structures

D4-1-86. Develop criteria and protection adjacent to railway tracks.

This part is ready for publication in the bulletin.

D4-2-87. Study AAR bridge program and make recommendations for implementation at FAST.

This assignment is dependent on the results of the Bridge Research Workshop testing and reports. They will review the reports and then make recommendations.

D4-3-88. Develop specifications for inspection, evaluation and repair of damaged prestressed concrete.

This written part was not ready for subcommittee letter ballot at our May meeting but may be ready by our February 1990 meeting. One consideration is to incorporate this information in Part 14 and submit the rewritten Part 14 as the results.

D4-4-88. Develop criteria for prevention of scour damage to piers.

The subcommittee has begun reviewing material in the Manual (located in four different Parts) and will attempt to tie all this information together as a single part.

Subcommittee 5. Design of Concrete Components for Bridges in Seismic Zones

D5-1-89. Develop design criteria or retrofit specifications or guidelines.

This is a new subcommittee established to begin reviewing material in the Manual and from other sources for the purpose of establishing a completely new part which will aid in development of specifications or guidelines which relate to new concrete bridge component design or retrofit of the existing components. A questionnaire was sent out to the Class I railroads and other railroads which may be in high seismic zones. Not all of the responses have been received, therefore, the results have not been tabulated.

Subcommittee 6. Design of Foundations

D6-1-89. Review and rewrite Part 4, Pile Foundations.

This part has passed letter ballot by the committee and should be ready for publication in the bulletin shortly.

D6-2-89. Review and rewrite Part 22, Specifications for Subsurface Investigation.

This part is in the process of being rewritten. The rewritten part was not ready for letter ballot by subcommittee at our May meeting but should be ready by our February 1990 meeting.

Excerpts From Annual Report of A.R.E.A. Committee 9 - Highway-Rail Crossings

K. J. Anderson, Chairman

Status of Subcommittee Assignments:

Subcommittee A. Recommendations for Future Study and Research

This subcommittee is responsible for the development of topics which should be addressed in the Manual. Mr. Shoemaker has just recently taken over the responsibilities of Mr. Bob Cutter, who has indicated that he will no longer be able to attend committee meetings.

Subcommittee B. Revision of Manual

This subcommittee is responsible for the processing of Manual revisions through the committee. It is likely that some revisions will come from Subcommittee D-1-87 next year. Mr. Zeinz has just recently assumed the responsibilities of this committee from Mr. Allen Moore who may not be able to attend future meetings.

Subcommittee D-1-87. Foundations for Highway-Railway Grade Crossings

A draft manual revision for Chapter 1, part 1, has been submitted to other members of the subcommittee for review. The entire committee will have the opportunity to review the proposed changes at our next meeting. Our goal is to recommend manual changes on this topic next year. Also, we are soliciting a volunteer to become Chairman of this subcommittee. Due to changes in job responsibilities, Mr. Walker has been unable to attend recent meetings.

Subcommittee C-1-87. Grade Crossing Surfaces

This subcommittee has been delegated the responsibility to work together with ASTM in development of standard material specifications and product performance specifications for crossing surface material. This subcommittee is also studying the proper location of joints in the vicinity of the crossing. Tom Zeinz will probably need some assistance from the AAR in developing this information.

Subcommittee D-2-87. Approaches to Highway-Railway Grade Crossings.

This subcommittee is currently attempting to address the "hump-back" crossing. Data is being collected to develop minimum undercarriage clearance requirements based on a specific design vehicle. The National Committee for Uniform Traffic Control Devices is studying the use of a sign to designate restricted vehicle use at high profile crossings.

Subcommittee D-3-87. Grade Crossings and Separation Elimination.

Their are several reports available which adequately address these topics. Mr. Shoemaker will prepare a summary of the information currently available. Upon completion the summary will be submitted for printing in the manual as information. Once this is accomplished, hopefully next year, this subcommittee will be discontinued.

Excerpts From Annual Report of A.R.E.A. Committee 10 - Concrete Ties

A. N. Hanna, Chairman

Subcommitee Assignments:

Subcommittee A—Recommendations for Future Study and Research

The committee recommends that a research study be initiated to evaluate the behavior of concrete ties under impact loading. This study should include both laboratory and field testing as well as reinforced and prestressed concrete ties. It is proposed that funding for this study could be sought from the National Science Foundation (NSF).

Subcommittee B—Revision of Manual

Subcommittee has recommended changes to Articles 1.2.3.1 and 1.2.3.2 of the Manual. The proposed changes were sent to all committee members in a letter ballot. All responding members accepted the changes. The recommended changes were submitted for Board approval.

Subcommittee 1. Flexural strength.

C1-1-87. Monitor developments in prestressed and reinforced concrete technology which may effect concrete tie requirements.

D1-2-89. Investigate impact resistance and design requirements for concrete ties.

D1-3-87. Evaluate acceptance criteria for repeated load test.

The subcommittee recommended that axle load should be considered a factor in developing concrete tie strength requirements, and therefore it will be included in Task C1-1-87. Task D1-2-89 is a new assignment and Task D1-3-87 has been recently assigned to this subcommittee. Work on these two tasks has just begun. Because of the importance of these activities to the development of acceptable concrete ties, it is recommended that these activities be continued during the next year and that subcommittee assignments be revised in the future as needed.

Subcommittee 2. Switch, Bridge and Crossing Ties

D2-1-89. Investigate requirements for concrete switch ties, bridge ties, and grade crossing ties.

The subcommittee has defined a work scope to include the following three items:

1. Development of recommended practice for the design and manufacture of concrete turnouts.

2. Investigation and reporting on the economics and potential use of concrete ties in turnouts.
3. Development of guidelines for use of concrete ties in grade crossings.

As the use of bridge ties is currently very limited, work on this topic has been temporarily postponed. Therefore subcommittee's work will address turnout and grade crossing ties only at this time. Because of the increased interest in the use of concrete turnouts, we recommend that these assignments be continued until the subcommittee develops appropriate recommendations for incorporation in the Manual.

Subcommittee 3. Fastenings.

D3-1-83. Revise current test requirements.

D3-2-83. Investigate the effect of axle loads and tie spacing on fastening requirements.

D3-3-86. Review and recommend revisions of the load magnitude specified for the fastening repeated-load test.

Tasks D3-1-83 and D3-2-83 involve consideration of many variables and alternatives, and therefore a considerable effort will be required to complete these tasks. The subcommittee is currently drafting a questionnaire for submission to the railroads that use concrete ties to advise on items that should be changed in the current Manual recommendations. Results of the questionnaire will be used in developing revised and simplified requirements for concrete tie fasteners. Task D3-3-86 has been recently assigned to this subcommittee and is currently under study. In view of the need to develop better guidelines for rail fasteners, we recommend that this subcommittee's assignments be continued until appropriate simplified recommendations are developed to replace the current Manual recommendations.

Subcommittee 5. Maintenance Requirements

C5-1-87. Maintenance requirements of concrete ties, including pads and insulation.

The subcommittee has prepared a questionnaire for review by AREA headquarters and submission to those railroads that have concrete tie tracks. In view of the increased usage of concrete ties, we feel that this should be an ongoing assignment. Results of the questionnaires will be used in developing Manual recommendations for maintenance requirements of concrete tie track.

Subcommittee 6. Ballast Requirements

C6-1-87. Collaborate with Committee 1 on concrete tie ballast requirements.

The subcommittee has continued collaboration with Subcommittee 1 and will continue to furnish data and recommendations concerning concrete tie ballast requirements. Committee 10 will retain the ballast requirements for concrete tie track in Chapter 10 until it is adequately covered in Chapter 1. We recommend that the subcommittee continues this liaison activity to insure appropriate coverage of ballast requirements for concrete tie track in Chapter 1 of the Manual.

Excerpts From Annual Report of A.R.E.A. Committee 11 - Engineering Records & Property Accounting

R. M. Davis, Chairman

Program of Subcommittee Assignments

Subcommittee A—Recommendations for Further Research and Study

Vice Chairman Ranuio spent much of his time assisting other subcommittee chairmen during the year. At the same time consideration was given to further study on the following issues:

- Flat rate billing for I&C projects
- Mechanized billing for I&C projects
- Elimination of 49 CFR, Part 1262, Uniform System of Records of Property Changes for Railroad Companies
- The use of software packages
- Forming a separate subcommittee for depreciation studies

Subcommittee B—Revision of Manual

The complete rewrite of technical chapter 11 was completed, officially sanctioned by Committee 11 and included in the AREA Manual. Two subjects not covered in the rewrite (accounting for rolling stock - highway equipment and Canadian rules for property accounting) have now been reduced to writing and are in the approval process by Committee 11 membership. We are planning for inclusion in the next update of the AREA manual.

After we completed our documentation regarding Canadian accounting rules, the National Transportation Agency (NTA) of Canada published a proposal to change the Canadian accounting rules for programmed track rehabilitation. The NTA has solicited comments from railroads, the accounting profession etc. regarding these proposed changes. It will be a long time before this issue is resolved. Rather than change our plans for the Manual additions, we plan to process as written. We will address the changes for track accounting in Canada if and when they happen.

Subcommittee 1—Accounting

See report for Subcommittee B above.

Subcommittee 2—Office and Drafting Practices

Under the guidance of Vice Chairman Ranuio, Subcommittee 2 is researching current systems on all class 1 railroads.

Subcommittee 3—Taxes

The analysis of the major ramifications of federal tax legislation is continuing. The consequences of FAS 96, revised accounting treatment of deferred taxes, are still being explored as are the following subjects.

- The corporate alternative minimum tax
- 50 year life for grading and tunnel bores
- Accounting for units of property under RRB

Subcommittee 4—Planning, Budgeting and Controls

This subject continues to engender interest and the enrollment of new members in Committee 11. New subjects include some interesting things being done by the Santa Fe in the area of electronic review and approval of Authorities for Expenditures (AFEs). Subcommittee 4 will look into this.

We have resolved the question of overlapping responsibility with Committee 22. We have been in communication with Committee 22 (verbal and written) and the subject of overlap is no longer an issue.

Excerpts From Annual Report of A.R.E.A. Committee 12 - Rail Transit

D. D. Dali, Chairman

Status of Subcommittee Assignments:

Subcommittee C-1-87. Rail Corridor Evaluation

Draft of chapter section in progress. Assignment 70% complete.

Subcommittee C-2-87. Special Trackwork and Roadway Considerations

Questionnaire responses being evaluated.

Subcommitee C-3-87. Special Bridge and Structural Considerations

Draft of station design guidelines on ballot.

Subcommittee C-4-87. Rail Transit Electrification, Collaborating as Necessary with Committee 33

Committee membership being developed.

All SC assignments are ongoing until initial chapter manual is published.

Excerpts From Annual Report of A.R.E.A. Committee 13 - Environmental Engineering

R. C. Brownlee, Chairman

Status of Subcommittee Assignments:

Subcommittee 1—Water Pollution Control

Subcommittee has three ongoing assignments at this time. Assignment C1-1-85—Leaking Underground Storage Tank Summary of Rules, has been delayed in the past due to the failure of EPA to promulgate these rules. The UST regulations have now been completed and we estimate completion of this assignment in 1990. Assignment C1-2-86—Report on Stormwater Regulations, has similarly been delayed due to failure of EPA to promulgate regulations. These regulations still have not been published by EPA, therefore work continues to be delayed. Assignment D1-3-88—Investigation and Control of Groundwater Contamination, is being conducted jointly with Subcommittee 3. Work on this assignment is 95% complete with estimated completion in 1990.

Subcommittee 2—Air Pollution Control

A new subcommittee chairman has been appointed and membership of the committee is being reorganized. Assignment D2-1-84 Revision of Manual, is approximately 50% complete with estimated completion date of 1991. We expect work to proceed on this assignment with recent reorganization of the subcommittee.

Subcommittee 3—Land Pollution Control and Solid Waste Management.

The committee as a whole has agreed that many of the concerns of Subcommittee 3 are directly related to contamination or prevention of contamination of groundwater, therefore Subcommittees 1 and 3 are jointly working on Assignment D1-3-88. As stated above, this assignment is approximately 95% complete with estimated completion date of 1990.

Subcommittee 4—Noise Pollution Control

Subcommittee 4 is currently inactive with members assigned to other subcommittees.

Subcommittee 5—Plant Utilities

Assignment D5-1-82—Revision of Part 5 Material in the Manual, is progressing with some sections to be considered for manual inclusion during 1990. The total assignment is approximately 85% complete with estimated completion date of 1990. Assignment D5-2-86—Environmental Considerations for Locomotive Fueling Facilities, was dropped by the committee since this assignment related directly to mechanical appliances which are outside the expertise of the current committee. Pollution control facilities associated with locomotive fueling is being considered under Assignment D1-3-88.

Excerpts From Annual Report of A.R.E.A. Committee 14 - Yards and Terminals

J. S. Acs, Chairman

Status of Subcommittee Assignments:

Subcommittee A

Presently there are a number of subjects under evaluation to determine the validity of future considerations:

1. Automatic Car Identification System.
2. Track Geometry concept in Terminal/Yard Operation.
3. Modernization of Fueling Facilities.
4. Coach Yard support facilities for Commuter/Passenger Operation
5. Regional/Short Line Railroads effect on Yard/Terminal concept.
6. Yards effect on run through trains.
7. Environmental consideration relative to Yards & Terminals.

Subcommittee B

The proposed consolidation of Parts 3 and 4—Chapter 14 (Freight Delivery & Transfer and Specialized Freight Terminals) have been approved as manual material. Voting is presently in progress.

Subcommittee D-1-74—Bulk Material Handling Systems

The report has been published as information and is now being modified to be submitted as manual material. Voting is presently in progress.

Subcommittee C-1-82—Intermodal Terminals

The original material has been published in the manual. Presently we are updating this subject to incorporate the handling of doublestack equipment.

Subcommittee D-8-88—Bulk Fluid Transfer Facilities

This subcommittee has had a slow start. The preliminary report will be available early in 1990. The final communique as information will be available late 1990 and update of the manual in 1991.

Subcommittee D-3-84—Run Through Trains Effect on Yards

This study began in 1983 under the direction of four (4) different chairmen. The present chairman progressed the work to a preliminary report, however he will also anticipate to resign prior to the work completion. Replacement is presently being solicited.

Subcommittee D-4-86—Control of Cars with Contaminated Wheels

The subcommittee made some progress with changes initiated by the second chairman at the helm. We anticipate to report for information in the latter part of 1990.

Subcommittee C-2-88—Working with TRB on Intermodal Terminals

This is an on-going effort to coordinate AREA/TRB activities within the set guidelines of both organizations. This is a one-man subcommittee basically in an advisory capacity to report and monitor TRB activities and coordinate periodical conference attendance with our committee members.

Subcommittee D-6-87—Design of Reclamation Plants

The final report is presently being reviewed by the full committee. The report will be submitted as information by the end of this year.

Subcommittee D-7-87—Yard Control Systems

The chairman has not submitted any report since the subcommittee's inception. However, the initial draft is now under preparation and will be reviewed at our November 1989 meeting.

Subcommittee D-2-89—Rail/Water Transfer Facilities

This is a new subject assignment. We expect the initial report to be available by June 1990, Bulletin distribution by December 1990 and Technical Manual format by March 1991.

Excerpts From Annual Report of A.R.E.A. Committee 15 - Steel Structures

E. Bond, Chairman

Brief Status of Subcommittee Assignments:

A. Recommendations for Further Study and Research

The Committee is working closely with the AAR bridge research project through John Choros of the AAR lab.

B. Revision of Manual

The committee has been working to prepare this year's submittal for changes to chapter 15 of the AREA Manual and is working closely with other subcommittees on proposed future changes.

C-1-60. Bibliography and technical explanation of various requirements in AREA specifications relating to iron and steel structures.

There is a continual need for this committee to update and add items in its area of responsibility. They have recently been concentrating on a solution that will eliminate the many questions the committee receives concerning the 100/S portion of the impact formula.

C-2-82. Fracture Control Plan

The subcommittee has developed a fracture control plan, now included in the manual. It needs to continue as there will be a continuing need to update the fracture control plan.

C-3-86. Steel fabrication—materials, methods, quality control procedures and qualifications of fabricators.

The committee continues to work on the development of specifications for loading details for fabricated members and other recommended changes in the fabrication area.

D-1-79. Develop specifications for the earthquake design of steel railway bridges.

The subcommittee has submitted it's recommendation for an addition to the manual to cover earthquake design forces. Recommend this assignment be considered.

D-2-87. Develop specifications for the design of confined elastomeric bearings in collaboration with Committee 8.

The subcommittee continues work on developing specifications for Manual inclusion. All work for development of pot bearing specifications will be done by this subcommittee per agreement with Committee 8.

D-3-87. Obtain data from which the frequency of occurrence of maximum stress in steel railway bridges may be determined under service loading.

This subcommittee has a very difficult task and has been continuing to work with the bridge rating group. More test data is needed, but most of this will come from the AAR Bridge Research project.

D-4-88. Methods for repairing damaged steel bridge members.

There is much interest in this new subcommittee which has made some progress with its assignment. It is estimated that approximately three more years will be needed to complete this particular assignment.

Excerpts From Annual Report of A.R.E.A. Committee 16 - Economics of Plant, Equipment and Operations

C. Bach, Chairman

Brief status of each subcommittee assignment:

Subcommittee B—Revision of Manual

Information that was received from AAR on the Davis Formula is being reviewed prior to being incorporated in a revision to the manual. This is a continuing assignment.

Subcommittee 1:

D-1-84. Economics of Train Speed

The final report was published in Bulletin No. 721 Vol. 90 (1989). As this completes this assignment, approval was given to terminate this assignment.

Subcommittee 2:

D2-1-83. Economics of Automated Train Inspection Equipment and Location

A second draft of the report is being circulated for comments. The committee will be reviewing the report at their next meeting and expect that it should be finalized for submission for publication in early 1990. The report will give the benefits that will be obtained by using automatic inspection equipment to improve safety and reduce cost of operations.

Subcommittee 3: Application of Industrial Engineering to the Railway Industry

D3-1-87. Railway Application of Artificial Intelligence

Several railways have indicated that significant improvements have been obtained using A.I. systems. The committee is preparing an industry survey for AREA board approval in order to obtain information on the extent A.I. methods are being used and benefits that have been obtained in railway operations. The results of the survey will be published for the information of AREA members. The survey is expected to be completed in 1990 and publication of information will be made soon after the results have been received.

Subcommittee 4:

D4-1-87. Economics of Railway Operations without Institutional Restraint

Material gathered to date was transferred to the new Chairman, and he has been sorting through the data to enable him to formulate a way to structure the study and the report that will be produced. When complete the report will give railways valuable information on how they could reduce their cost of operations if they are prepared to overcome accepted and imposed institutional restraints.

Subcommittee 5:

D5-1-87. Factors to be Considered in Evaluating Advance Train Control System.

The Committee had been reviewing the testing work and studies that are being conducted by various railroads in the area of ATCS. A second draft of the report was prepared and distributed for comment. The report is expected to be finalized early in 1990 for submission for publication. The report will greatly assist railways in their evaluation of using ATCS on part or all of their operations.

Excerpts from Annual Report of A.R.E.A. Committee 17 - High Speed Rail

R. D. Johnson, Chairman

Status of Subcommittee Assignments:

Subcommittee D-1-88. High Speed Rail Corridor Evaluation, Including Terminal Considerations.

A. Assigned January 26, 1989

B. Progress to Date: Suggest title change to be "High Speed Rail Corridor Evaluation." Corridor evaluation items needing identification were listed and a brief introduction and outline are targeted for initial report by mid-March 1990 on the following by named members:

1. Demand for Service and Future Market—Jean Paré
2. Consideration of Alignment and Location—Tom Gray
3. Operational Analysis (Types, Frequency, etc.)—Bob Wallendorf
4. Analysis of Cost Factors—Ken Felsburg

5. Environmental Factors—Bob Schmelz
6. Institutional Factors—Dick Hill

Following evaluation items need to be addressed also but need leadership volunteers/additional members:

1. Evaluation of Terminal and Station Location
2. Consideration of Auxiliary Facilities and Associated Land Requirements
3. Consideration of Financing Alternatives

A current list of Worldwide High Speed Rail Projects will be prepared and updated regularly.

C. Completion Date: To early to determine.

D. Benefits: A standard of evaluation criteria which does not now exist.

E. Problem Areas: None

F. Recommendations: Continue

Subcommittee C-1-88. Track Considerations for High Speed Rail Including Loads on Structures, Collaboration with Appropriate Vehicle Groups and Committees 4 - Rail and 5 - Track.

A. Assigned: January 26, 1989

B. Progress to Date: Suggested title change to be "High Speed Rail Track, Structure, and Track-Train Interaction." Various existing track specifications and maintenance parameters for the construction and maintenance of High Speed Rail Operations are being collected for reference purposes. Classification and relevancy of this information will occur for future use and a storage matrix method must be developed for ease of future usage. A preliminary statement of track specifications will be developed from this action.

C. Completion Date: Too early to determine.

D. Benefits: A standard of evaluation criteria which does not now exist.

E. Problem Areas: None

F. Recommendations: Continue

Subcommittee C-2-88. Signal and Electrification Considerations for High Speed Rail, Collaborating with Appropriate Vehicle Groups and Committee 33—Electrical Energy Utilization

A. Assigned: January 26, 1989

B. Progress to Date: Suggested title change to be "Vehicles, Control, and Propulsion System Considerations for High Speed Rail, Collaborating on Electrification with Committee 33—Electrical Energy Utilization and the AAR Communication and Signal Division."

Finalization of a report for Informational Publication of a Survey of Wheel on Rail Traction Vehicles Capable of Running at 200 Km/Hr or More is underway. Subcommittee subdivided into three groups for study purposes as follows with identified Chairman with initial report drafts targeted for October 5, 1989.

1. Vehicles—Joe Schmidt
2. Control Systems/Signals—Bill Matthews
3. Propulsion/Electrification—Peter Shaw

C. Completion Date: Too early to determine.

D. Benefits: A standard of evaluation criteria which does not now exist.

E. Problem Areas: None

F. Recommendations: Continue

Subcommittee D-2-88. Evaluation of Systems for Operation Over 250 MPH

A. Assigned: Not as yet assigned for study.

Excerpts From Annual Report of A.R.E.A. Committee 22 - Economics of Railway Construction & Maintenance

W. C. Thompson, Chairman

Status of Subcommittee assignments:

Subcommittee B.
Revision of Manual.

(a) Assigned—Unknown

(b) Progress—Rewrite of entire Manual underway with progress of each part shown below.

(c) Completion date: 6/92 per Committee's five-year goals.

(d) Benefits—Greatly improved quality of information in Chapter 22 of Manual.

(e) Problems—None

(f) Recommendations—Continue.

Subcommittee B-1, Part 1
Revisions of Manual—Personnel.

(a) Assigned—1/14/88

(b) Progress 90%—Draft complete and approved for vote

(c) Completion date: 6/91

(d) Benefits—Improved information.

(e) Problems—None

(f) Recommendations—Continue.

Subcommittee B-1, Part 2
Revisions of Manual—Programming Work and Budgeting.

(a) Assigned—1/14/88

(b) Progress 90%—Draft complete and approved for vote

(c) Completion date: 1/91

(d) Benefits—Provide high quality information.

(e) Problems—None

(f) Recommendations—Continue.

Subcommittee B-1, Part 3
Revisions of Manual—Construction and Maintenance Operations.

(a) Assigned—1/14/88

(b) Progress 20%—Outline complete

(c) Completion date: 6/91

(d) Benefits—Improved information

(e) Problems—None

(f) Recommendations—Continue.

Subcommittee B-1, Part 4
Revisions of Manual—Equated Mileage Parameters.

(a) Assigned—1/14/88

(b) Progress 95%—Draft complete and approved for vote.

(c) Completion date: 6/91

(d) Benefits—Improved usefulness and understanding of equated mileage parameters.

(e) Problems—None

(f) Recommendations—Continue

Subcommittee D-1-87. Develop Economics of Methods to dispose of scrap and obsolete materials.

(a) Assigned—1/26/88

(b) Progress 100%—Published

(c) Completion date: 1/89

(d) Benefits—Addresses an important and extensive topic.

(e) Problems—None

(f) Recommendations—None

Subcommittee D-2-86. Economics of Various Surface Gang Consists used by Railroads in North America.

(a) Assigned—1/14/86

(b) Progress 10%

(c) Completion date: 6/90

(d) Benefits—Information

(e) Problems—None—New chairman assigned.

(f) Recommendations—Continue to completion.

Subcommittee D-3-88. Economics of Vegetation Control Methods Working with Committees 1 and 13.

(a) Assigned—1/14/88

(b) Progress 30%

(c) Completion date: 6/90

(d) Benefits—Develop costs of Vegetation Control Methods.

(e) Problems—None

(f) Recommendations—Continue.

Subcommittee D-4-87. Economics of AREA Standard vs. High Strength Rail.

(a) Assigned—6/87

(b) Progress 98%—Approved for publication.

(c) Completion date: 6/89

(d) Benefits—Develop rail usage priority and economics.

(e) Problems—None

(f) Recommendations—None.

Subcommittee D-5-87. Economics of Standardization of Turnout Materials.

(a) Assigned—1/87

(b) Progress 50% complete. Draft report being prepared.

(c) Completion date: 6/90

(d) Benefits—Can have significant impact on industry.

(e) Problems—None

(f) Recommendations—Continue.

Subcommittee D-6-88. Economics of Rail Grinding.

(a) Assigned—1/14/88

(b) Progress 100% complete at March conference.

(c) Completion date: 3/89

(d) Benefits—Present latest information on Rail Grinding.

(e) Problems—None

(f) Recommendations—Continue

Subcommittee D-7-88. Economics of Various Fixations of Rail to Wood Ties.

(a) Assigned—1/14/86

(b) Progress 100%—Published

(c) Completion date: 6/88

(d) Benefits—Review and analyze impact of various fastening systems.

(e) Problems—None

(f) Recommendations—Continue

Excerpts From Annual Report of A.R.E.A. Committee 24 - Engineering Education

J. W. Orrison, Chairman

Status of Subcommittee Assignments:

Subcommittee A—Recommendation for further study and research

Assigned: Prior to 1983

Progress: Subcommittee A will be involved in assisting with review of establishing subcommittee goals for the next 2 years and will be responsible for identifying topics that should be handled as recommendation for further study and research.

Completion: Should be continuous.

Benefits: Develop meaningful new assignments.

Problems: None

Recommendations: Continue

Subcommittee C-1-82—Recruiting and Speakers

Assigned: Prior to 1983

Progress: Publish an annual survey of college graduate hiring by railroad engineering and maintenance departments. Efforts to make the report more current now allow publication of the report in the spring. Advise schools of availability of railroad speakers for student groups.

Completion: Should be continuous.

Benefits: Provide starting salary information to the railroads. Inform top students of career opportunities in railroading.

Problems: None

Recommendations: Continue

Subcommittee C-2-84—Undergraduate Research Fellowship Program

Assigned: 1989

Progress: The purpose of this subcommittee has been redirected to administrate the Undergraduate Research Fellowship Program. The URFP involves announcement, judging and selection of up to 2 Fellowships for the study of rail related topics.

Completion: Should be continuous.

Benefit: Opportunity to increase student interest in Railway Engineering by working on research projects and allows AREA to provide the results of the study to our membership.

Problems: None

Recommendations: Continue

Subcommittee C-3-85—Curriculum Development

Assigned: 1985

Progress: The purpose of the subcommittee is to provide opportunity for the railway industry to assist in the development of engineering curricula which might relate to railroad needs.

Completion: Expected in 1990.

Benefits: Opportunity to develop increased interest in railway engineering on the part of engineering faculty.

Problems: Low Activity

Recommendations: Continue

Subcommittee C-4-86—Student Relations

Assigned: Prior to 1983

Progress: Assigned in the administration of the first URFP. Will progress all Student Relations programs including the Student Affiliates.

Completion: Should be continuous

Benefits: Increased student and faculty awareness of AREA and the railroad industry.

Problems: None

Recommendations: Continue

Subcommittee C-5-71—Continuing Education

Assigned: Prior to 1983

Progress: Exploring new alternatives to stimulate AREA participation in continuing education.

Completion: Should be continuous.

Benefits: Provides opportunity for AREA members to update their knowledge of subject matter which is relevant to the railroad industry.

Problems: Low activity

Excerpts From Annual Report of A.R.E.A. Committee 27 - Maintenance Of Way Work Equipment

J. J. Hannaford, Chairman

Subcommittee assignments:

Subcommittee A—Recommendations for Further Study and Research

The task of this subcommittee is directing the committee toward new tasks, set goals and interface with the REMSA group.

Subcommittee B—Revision of Manual

Specifications for on-track roadway machines were approved by the board and published in 1989. The next phase of the clean-up is underway, with each item in the Table of Contents being updated.

Subcommittee C-1-74—Reliability Engineering, as Applicable to Work Equipment

A draft report on the questionnaire sent to all major railroad and railroad suppliers was submitted to Committee 27 at our March 89 meeting. The consensus of the committee was that the report was not ready for publication and will be re-done in a different format. The areas of concern highlighted in this report will govern the future direction of this subcommittee.

Subcommittee C-2-83—Preventative Maintenance of Maintenance of Way Equipment

Preventative maintenance schedules for 10 major pieces of maintenance equipment have been sent to all members for review. Discussion and revisions of these will take place at our August meeting in Chicago.

Subcommittee D-2-86—Computer Applications as Applicable to Work Equipment

This committee continues to gather information on computer assist packages for presentation to the committee.

Subcommittee C-3-84—Maintenance of Way Equipment Safety

This subcommittee has been inactive due to the resignation of its Chairman. A Chairman for this committee will be appointed at our August meeting in Chicago.

Subcommittee D-1-77—Training Programs for Machine Operators & Maintainers

This subcommittee is actively engaged in the standardization of training packages for machine operators. The railroad and construction industry was polled on availability of video tapes for training purposes. A large package of what is available has been sent to all committee members. The chairman of this subcommittee has resigned due to his retirement from railroad service.

Excerpts From Annual Report of A.R.E.A. Committee 28 - Clearances

J. T. Ormsby, Chairman

CONTINUING ASSIGNMENTS:

Subcommittee A. Recommendations for Further Study

Special study related to developing a standardized, central data base for storing RLC/ORER data, at the discretion of the carrier. No action taken at this time pending further discussion relating to data security and maintenance responsibility.

Subcommittee B. Revision of Manual

Committee is currently beginning upgrading graphics portion of chapter through computer graphics utilization.

Subcommittee C-1-62. Compilation of the Railroad Clearance Requirements of the Various States.

There have been no changes reported during the past year.

Subcommittee C-2-85. Compilation of a Comprehensive Glossary and Bibliography Pertaining to the Technical Literature on Railroad High and Wide Clearances.

There have been no changes reported in the past year.

Subcommittee C-3-86. Liaison Committee to coordinate with Short Line Railroads.

Despite the seeming importance of the topic, we have been forced to discontinue the study due to a lack of activity and an inability to chair the subcommittee. Few, if any, short line members show up at the committee meetings with any degree of regularity, and ideally the subcommittee should be chaired by a short line member. Possibly this study can be reopened at a later date under a more favorable environment.

Subcommittee C-4-85. Review of Railway Line Clearances to develop improved user accessibility.

At the present time a letter ballot is being prepared by the subcommittee chairman on a number of revisions proposed at last fall's meeting. The board will be notified of any recommended and ballot approved changes.

SPECIAL STUDIES:

Subcommittee D-3-85. Conversion of Heavy Capacity and Special Type Flat Car Section of the Official Railway Equipment Register (ORER) to UMLER Compatible Format.

With regard to cars with 10 or more axles, a number of car owners have noted errors in the UMLER car listings due to the fact that UMLER only provides a single character field for data, utilizing notes for indicating 10 or more axles. Our subcommittee chairman has been reviewing and correcting these errors with the UMLER people.

At such time as the work on Subcommittee D-4-85 is completed with its assignment, we propose combining these two subcommittee assignments as a single reviewing subcommittee with continuing status, assuring proper documentation of car diagrams, shear diagrams and tabulation of mechanical data with regard to new and proposed equipment.

Subcommittee D-4-85. Research and Develop Book Covering Heavy-Duty Car Diagrams and Ratings.

Since last report, this subcommittee has come under the chairmanship of J. L. Pearsall of the AAR. Because of his ready access to the information needed to wrap-up this area of study, the subcommittee has progressed measurably during the past year.

At the upcoming Philadelphia meeting in September, a work session is proposed which should bring this study within reach of completion. I wish to thank the board for their patience in waiting so long for the completion of an endeavor which initially was vastly underestimated in scope and depth.

Subcommittee D-5-86. Recommended Procedure for Safe Movement of Oversize and Overweight Shipments on Foreign (Jointly Operated) Track.

During the past year, the subcommittee met in Philadelphia for a work session at which time a tentative 'final' draft was written up for committee review at the September meeting. Pending the reception at the Philadelphia meeting, the final draft should be ready for ballot.

Subcommittee D-6-89. Recommendations for a Uniform Electronic Clearance Message.

This subcommittee has just opened during the past year. The chairman is one of our newest members and has already submitted his initial draft for review by the committee. It is not expected that this study should run into any unnecessary delays.

Excerpts From Annual Report of A.R.E.A. Committee 32 - Systems Engineering

R. B. Sliepka, Chairman

Report on Special (D) and Continuing (C) Assignments

Assigned Number	Description of Assignment	Date Assigned/ (Undertaken)	Estimated % Completion	Estimated Completion Date
C2-1-86	New applications computers	1-1-86	50%	5-91
C2-2-86	Data gathering techniques	1-1-86	80%	5-90
C3-1-78	Systems Eng. Design	1-1-78	50%	5-91
C3-2-78	Computer graphics	1-1-78	25%	5-92
D3-1-88	Toronto Symposium in 1989	1-1-88	95%	1-90
D4-1-86	Industry Standard	1-1-86	40%	1-91
D4-2-86	Manual Section Graphic Standards	1-1-86	25%	1-91

Excerpts From Annual Report of A.R.E.A. Committee 33 — Electrical Energy Utilization

A. J. Peters, Chairman

Subcommittee Assignments:

Subcommittee 1—Electrification Economics

C1-1-73. Monitor Economics of Electrification

This assignment was made in 1973 and is an ongoing review of the economic factors influencing the installation and maintenance of railroad electrification. A draft update for publication in the bulletin is in the process of being prepared.

Subcommittee 4—Railroad Electrification Systems

C4-1-76. Electrification Systems

This assignment was made in 1976 and consists of a review of the specification and guidelines for the major components and systems for railroad electrification, contained in Part 4 of Chapter 33 of the manual. The current activity consists of the development of specifications for foundations and footings for catenary structures. A number of new techniques and designs have been used in recent years which are worthy of review. This assignment was recommended by the committee for special emphasis during the last year. A revision to the manual is expected to be submitted before the end of the year.

D4-1-87. Safety Guidelines for Electrified Systems

This assignment was made in 1987 to review the safety requirements for the construction and operational maintenance of electrified systems. No progress has been made on this assignment to date. This assignment has been placed on hold awaiting a determination of the potential legal issues by the HQ staff.

Subcommittee 6—Power Supply and Distribution

D6-1-87. Review and Update Part 6 of the Manual

The first draft of a major update for the manual, reflecting major changes that have recently taken place in power supply technology, has been prepared and distributed for comments. The comments are currently being incorporated into a second draft.

Subcommittee 8—Equipment Generated Electrical Noise

D8-1-86. Review Part 8 of the Manual

This subcommittee was initiated in 1981 to look at the impact of electrical noise generated by railroad electrification equipment on other railroad equipment and determine whether a new section in Chapter 33 was required. This assignment has received low priority to date due to other committee assignments.

Excerpts From Annual Report of A.R.E.A. Committee 34 — Scales

F. J. Loyd, Jr., Chairman

Subcommittee Activities:

Subcommittee A—Recommendations For Further Study and Research

This is an ongoing subcommittee.

This Committee investigates and reviews products and technological changes in the Industry and makes recommendations to the Committee Chairman concerning need for further study and research.

The Kaman Science Corporation System and Kilo-Wate Weighing Systems, both being marketed throughout the U.S. have been referred to Subcommittee C-2-82 for study.

Two items identified by the Committee to be addressed pertain to: (1) Railroad Master Scale Program and (2) Recommended Track Scale Testing Criteria. A request has been made for the Board to approve the formation of a subcommittee to study Item 2, beginning in 1990.

Subcommittee B—Revisions of AAR Scale Handbook

This is an ongoing subcommittee.

Three changes were submitted to the membership for vote and all three approved. A fourth item was submitted to the membership for comment with a large percentage not in favor of the proposal.

This Subcommittee is one of the most important within Committee 34 because of the critical need to keep the AAR Scale Handbook up to date with technological advances, State and Federal Laws, etc.

Changes to the Handbook were made available to Committee Members and holders of the publication.

Subcommittee C-1-85—Preparation of Subjects for Publication

This is an ongoing subcommittee.

Changes to the AAR Scale Handbook to bring it into conformance with NBS H44 (State and Federal Regulations), as well as several typographical corrections and approval changes, were published early in 1989. There are currently no outstanding assignments pending.

Subcommittee C-2-82—Innovations in Scales Used in Connection With Operations of Railroad

This is an ongoing subcommittee that investigates new technology and changes in the weighing industry that affect Railroads.

The Subcommittee is presently monitoring a Kaman Weigh-in-Motion System located on the Burlington Northern in Kansas City, Kansas and plan to study data for a Kilo-Wate Weigh-in-Motion System tested on the CSX. Another item being studied deals with high speed Coupled-in-Motion weighing as relates to accuracy and to the benefits to the Railroad industry.

Subcommittee D-1-83—Criteria for the Location of Coupled-in-Motion Track Scales

A number of this subcommittee's members are associated with the National Conference on Weights and Measures S&T Railroad Advisory Committee, which has developed considerable data and information concerning topographical situations for Coupled-in-Motion Systems. This information has been studied by the subcommittee and a subcommittee meeting June 15, 1989, in Nashville, Tennessee resulted in a 1st draft of guidelines for this subject.

A meeting of this subcommittee during the October, 1989 Committee 34 meeting will result in a final draft to be presented to the full committee. This study is projected to be complete in Spring, 1990.

Subcommittee D-2-87—Investigate Stenciling of Cars Using Coupled-in-Motions Weights

The file on this study was lost at the time of the death of the former Subcommittee Chairman in December, 1987. The present Chairman is accumulating the data again and should be in position to conclude this study by Fall, 1990.

Proposed Manual Revisions

The following proposed Revisions of the A.R.E.A. *Manual for Railway Engineering* have been recommended to the association by the technical committee responsible for each chapter after a letter ballot is approved by: (1) a two-thirds majority of the eligible members voting, and (2) by at least fifty percent of the total eligible voting members of the committee. They are being published here for comment of the general A.R.E.A. membership and any other interested parties. Comments should be sent to A.R.E.A. headquarters by March 1, 1990. These comments will be considered by the A.R.E.A. Board of Direction in deciding whether to give final approval for inclusion of the proposed changes in the Manual Revisions which go into effect August 1, 1990.

Proposed 1990 Manual Revisions To Chapter 1 — Roadway and Ballast

Section 3.4 on the "Prevention of Stream Erosion" has been revised and updated including a new section on Scour and methods of predicting scour. The remainder of section 3.4 was updated to remove obsolete and illegal protection methods, and to expand the description of the various methods to include selection criteria, advantages, and disadvantages of each type of protection.

3.4 PREVENTION OF STREAM EROSION

3.4.1 General

This section deals with the drainage of the ground surface by natural waterways and addresses possible remedies to problems associated to natural surface drainage, such as erosion and silting. Figures 1 and 2 provide different examples of potential erosion and silting problems.

The following recommendations should be viewed as guidelines to properly select a possible remedy, in the effect to control erosion and silting during construction and in present and future maintenance. However, installation of some of these methods may be viewed as artificial obstructions, so State approval should be sought before implementation. For areas in which the methods are applicable, the District Office of the U.S. Army Corps of Engineers should be contacted with regards to permit applications and the type of work to be performed.

3.4.2 Scour

3.4.2.1 Definition

Scour is the removal of stream bed, backfill, slopes or other supporting material by streams, tides, dredging, propeller backwash, etc. All piers and abutments erected in water must be protected from scour, with the realization that conditions may change from those at time of design. Although flood stages are not necessary for scour to occur, the greatest scour occurs during the largest floods.

There are two types of scour:

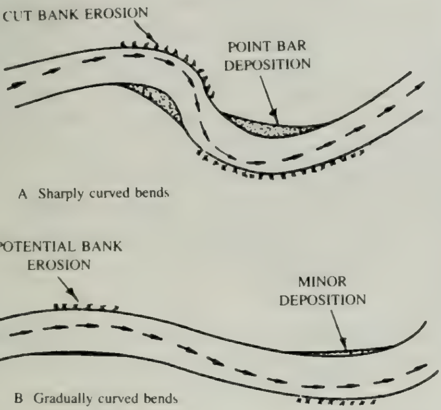
1. clear-water case—where material is removed from the scour-hole and not replaced;
2. sediment-transport case—where the scour hole is continuously supplied with the material from the sediment load carried on the stream bed.

3.4.2.2 General Description

Several different stream characteristics may be involved in scour. The stream channel may be moving, as in the migration of a meander. Degradation may be occurring as a natural geologic agent, or as a man-caused agent due to stream straightening, or by the reduction of sediment supplied (as by a dam). Degradation can expose bridge foundations to danger.

Aggradation can also occur. Deposition in the river causes a rise in water surface which can cause hang-up of drift and debris on the bridge. Those conditions constrict the stream flow and cause local scour.

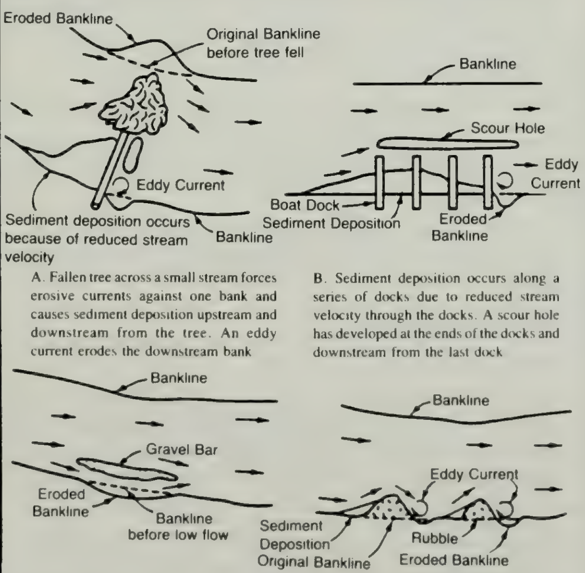
The embankment fills of a river crossing also can cause a contraction of the river in flood. This will force some flood plain flows laterally to the bridge opening. If flow returns to the channel upstream of the bridge, then there will be general scour over the waterway opening. If the flow returns along the embankment, then there can be severe scour at the abutment and, possibly, out to the first or second pier.



Sharply and gradually curved bends under normal flow conditions. Arrows indicate the path of the maximum stream velocity. During floodflow this path moves across the channel against the point bar.

FIGURE 1

REF. 1



A. Fallen tree across a small stream forces erosive currents against one bank and causes sediment deposition upstream and downstream from the tree. An eddy current erodes the downstream bank

B. Sediment deposition occurs along a series of docks due to reduced stream velocity through the docks. A scour hole has developed at the ends of the docks and downstream from the last dock

C. Erosive currents are deflected against bank by gravel bar exposed during low flow.

D. Irregular bankline caused by dumped rubble sets up eddy currents which erode bank. Sediment deposition is occurring upstream from the rubble

Obstacles in a stream can deflect current flow causing complex erosion and sediment deposition patterns.

FIGURE 2

REF. 1

There also is local scour at the piers and abutments. The geometry of the structures and the flow patterns are the determinants of this scour. The flow conditions are illustrated in Figure 3. Ordinarily, it is this local scour that is in the minds of bridge designers when they are formulating their designs.

In clear-water scour, the limiting condition occurs when there is no more capacity for transport out of the scour hole, i.e., when the boundary shown is equal to the critical tractive force of the bed material. This shear is a function of flow velocity, geometry, and roughness of the boundary. Critical traction is a function of the bed material. Thus, the depth of clear-water scour can be expected to be a function of geometry, flow velocity, and sediment size (or similar characteristic).

For the sediment-transport case, the limiting case occurs when transport out of scour hole equals sediment infilling. It is believed that neither velocity of flow nor sediment size should affect scour depth—if flow pattern is essentially unchanged, an unlikely event.

3.4.2.3 Methods for Estimating Scour

There are several methods that have been proposed for prediction, each in the form of an equation. The two most popular are presented below; others can be found upon review of the sources in the bibliography.

1) Laursen is best described by graph (see Figure 4).

2) Neville-sediment transport case

$d_s/b = 1.5\ (Y/b)^{0.3}$

where d_s = depth of scour from stream bed

b = width of pier

Y = depth of flow

The coefficient, 1.5, should change to 1.2 for round-nosed piers.

There is some feeling that the Laursen prediction is the major one that is applicable for the clear-water case. There is no field confirmation.

The Neville prediction appears to do the right things in the sediment-transport case.

As a generality, the depth of scour below stream bed is at least equal to width of pier or depth of flow in magnitude. Beyond this, there is an urgent need for field data so to allow creation of confidence in the prediction techniques.

Some consideration should be given to periodic underwater inspection to determine scour of potentially chronic areas. Annual soundings can also give a history of scour in different flow conditions.

3.4.2.4 Recommendations

3.4.2.4.1 General

Bridge piers should be located when possible, where there are minimal or no scour currents. It is not desirable to locate a pier immediately downstream of a sharp bend in the waterway. Straight alignments and right-angled crossings are desirable features for bridge locations. It *is* recognized that flood waters are faster where flood waters can overflow the channel. Piers located in deep, fast-moving water will be subject to more scour than piers in wider, slower streams.

To prevent undermining by scour, bottom of footing should be located at adequate depth below stream bed. Beds of silt or sand are more susceptible to deep scour, but piers on piles into sand and gravel have also been damaged by flood scour. Designs should minimize scour effects as much as possible.

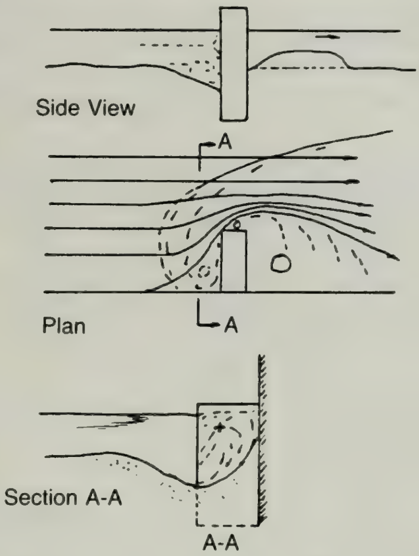


FIGURE 3 **REF. 3**

FLOW PATTERNS IN THE VICINITY OF THE SCOUR HOLE

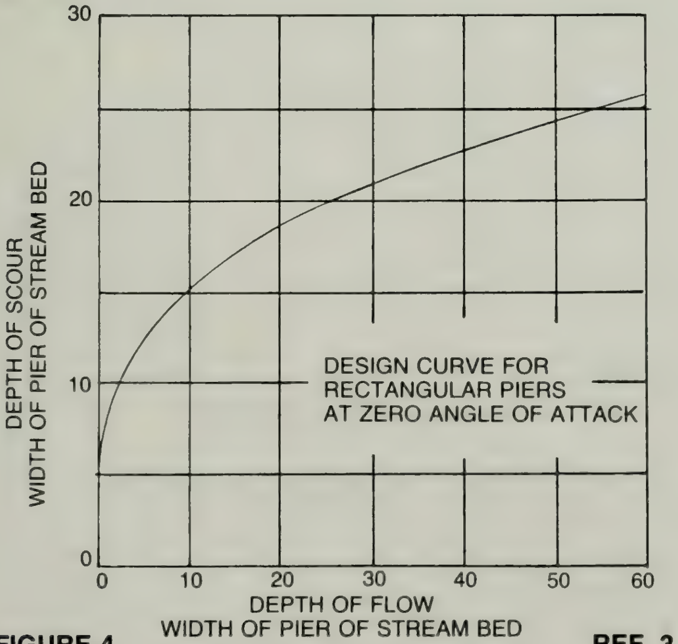


FIGURE 4 **REF. 3**

LAURSEN'S BASIC DESIGN CURVE FOR DEPTH OF SCOUR

3.4.2.4.2 Embankments Design

Clearings and borrow areas should not be placed directly upstream from a proposed embankment, as scour is increased markedly if they are so located.

The end-of-embankment slope from abutment to beyond first pier should be protected by rip-rap or sacked concrete.

Spur dikes projecting upstream from embankments cause the scour hole to be created upstream of the bridge location, reducing the effect upon the bridge pier. It would appear that an elliptical shape with a 2.5 ratio of major to minor axes provided good results. The dike itself requires rip-rap protection, at least on the upstream end.

Sheet-pile toe walls also have been used as protection. They do not inhibit scour, but they do provide restraint and prevent erosion of the embankment.

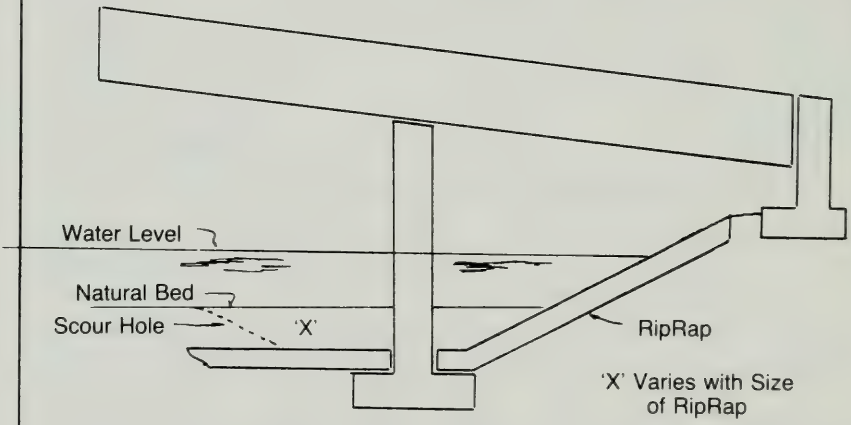


FIGURE 5 **REF. 3**

RIPRAP PROTECTION

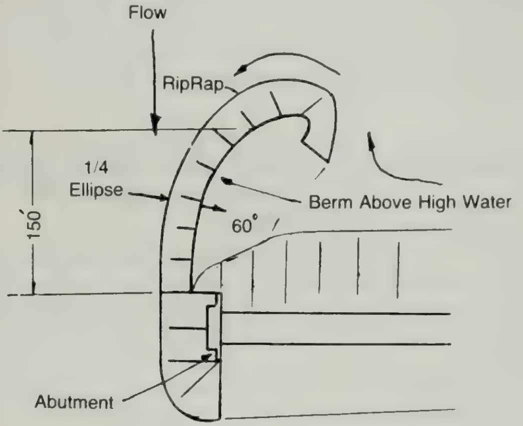


FIGURE 6 **REF. 3**

SPUR DIKE DETAILS

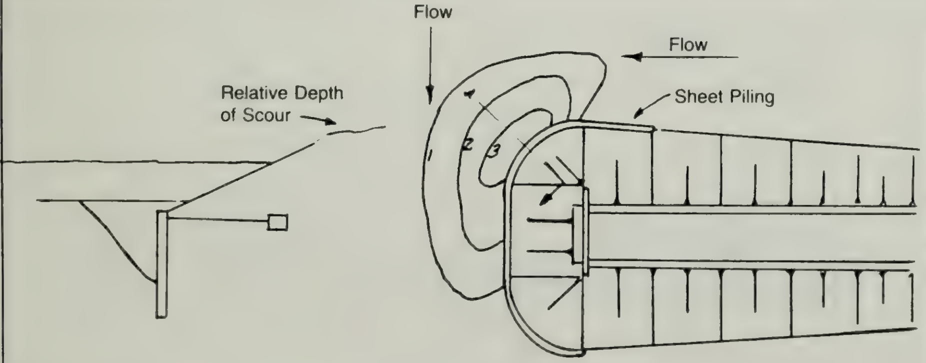


FIGURE 7 **REF. 3**

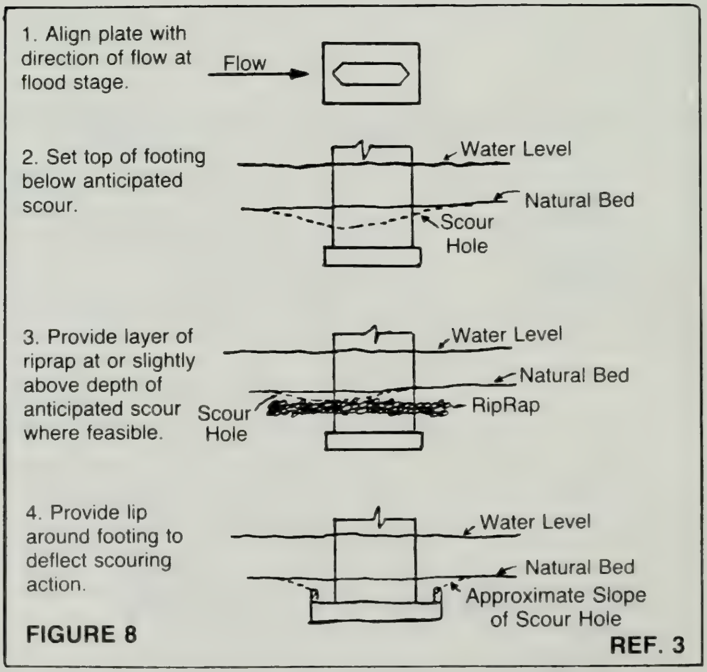
SHEET PILE EMBANKMENT PROTECTION

3.4.2.4.3 Piers Design

In alluvial material the tops of footings should be placed well below depth of estimated scour. The frequency of design flood will depend on the importance and cost of the structure. A very conservative depth is recommended when setting foundation elevations. Figure 8 shows some common sense practices.

Damage has been reported with piers founded on erodible shales, sandstones, and other rocks. The interface between footing and rock must be protected. No method, other than experience, is available to predict rock scour. Thus, prudence dictates extra measures to hydraulically streamline the pier base, and also, to place rip-rap protection around the base.

With pile foundations for piers, there often is an associated complacency or illusion of safety. The foundation must serve even after scour has occurred. The system may utilize batter piles or the reserve bending strength usually available for frame action. Large precast piles or composite piles (e.g., steel pipe filled with concrete) will, many times, improve ability to withstand scour effects. It must be remembered that design assessing axially-loaded piles assume the piles are surrounded by earth along their lengths, i.e., that they act as short compression members. Where scour exposes some lengths of piles, column action will occur and the piles will need to have been reinforced to withstand this loading. Several typical pile foundation designs are shown in Figure 9.



1. Align plate with direction of flow at flood stage.

2. Set top of footing below anticipated scour.

3. Provide layer of riprap at or slightly above depth of anticipated scour where feasible.

4. Provide lip around footing to deflect scouring action.

FIGURE 8

RECOMMENDED PRACTICE FOR SPREAD FOOTINGS

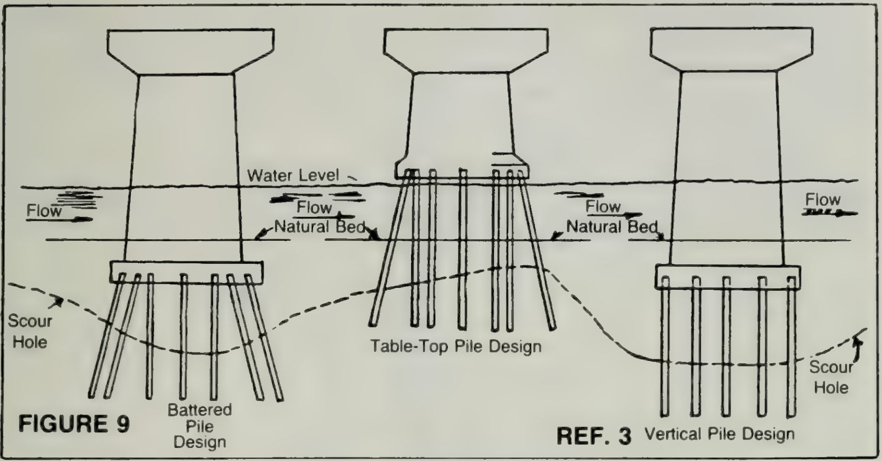


FIGURE 9 **REF. 3**

PILE FOUNDATION DESIGNS

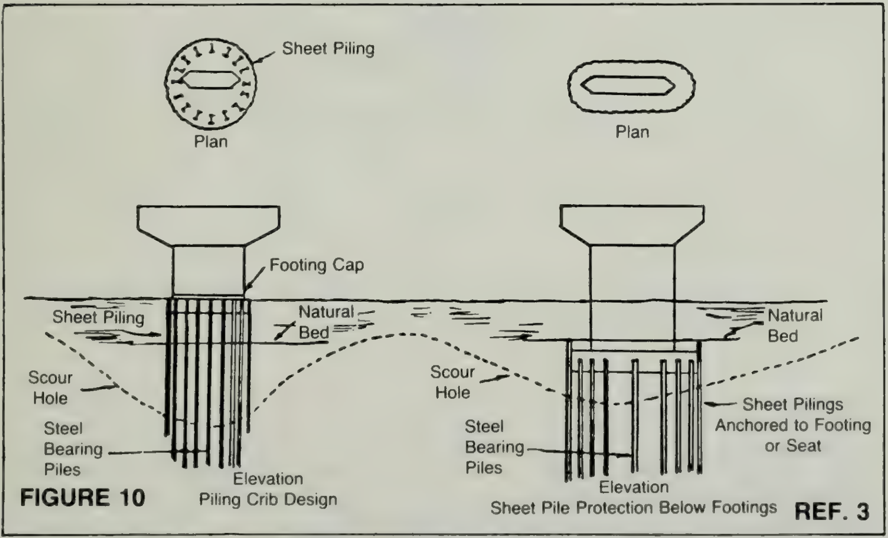


FIGURE 10

Elevation
Piling Crib Design

Elevation
Sheet Pile Protection Below Footings **REF. 3**

SHEET PILE PROTECTED DESIGNS

Where scour is expected below footing level, some protection may be afforded by sheet piling surrounding the footing. Two configurations for installation is shown in Figure 10.

3.4.2.4.4 Maintenance

Once scour has occurred, the course must be established. This can provoke appropriate measures, such as a change in stream alignment. Sometimes, remedial work can be detrimental, as shown in Figure 11.

Where erosion has occurred, it is common practice to place rock or rip-rap in the void or to protect the replaced soil with rip-rap, bagged concrete or wire-enclosed rock. The velocity of flow and direction of current are involved in selecting proper size of replacement material to avoid it being removed also. Sometimes, placing concrete underwater in addition to placing supplemental piles is necessary to protect the pier after scour has occurred.

It must be noted that post-flooding underwater inspection may not find a scour problem if the scour hole has been filled during flood recession. Most of the time the severity of such scour problems cannot be identified with techniques available today. There is an urgent need to be able to measure the area and depth extent of scour during flooding with reliable instrumentation.

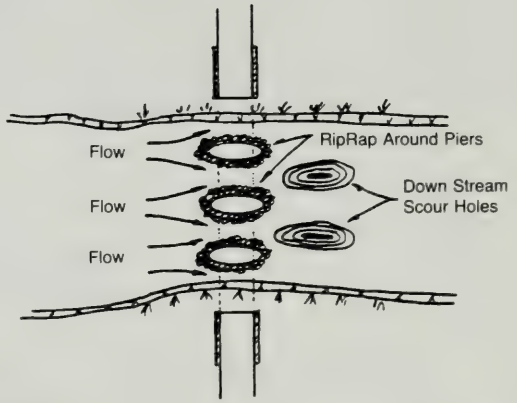


FIGURE 11

REF. 2

LOOSE RIPRAP PLACED AT PIERS TO PREVENT LOCAL SCOUR MAY BE DETRIMENTAL TO THE CHANNEL.

3.4.3 Dikes, Jetties and Groins

Dikes, jetties and groins fall into two (2) basic types, permeable and impermeable. The permeable type functions by slowing the velocity of the current of a sediment-carrying stream, thus slowing the erosion and causing a deposition of the suspended silt. In general, the heavier the sediment load, the more effective these structures will be. The impermeable type structure functions by physically turning the flow away from the erodable surface. Both types, as most commonly used, are discussed in more detail below.

3.4.3.1 Timber Pile Dikes

Timber piles dikes can be utilized on waterways carrying heavy sediment loads. The heavy sediment is essential for effective performance.

The typical installation includes several spurs constructed perpendicular to the bank and extending out into the channel with a longitudinal section, often termed a "trail dike," located at the channel end of the spur extending downstream. An example of these dike components is shown in Figure 12.

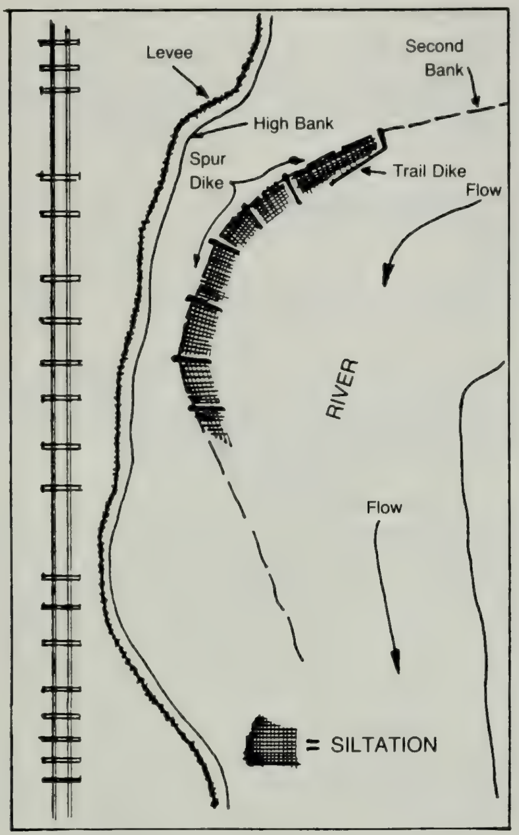


FIGURE 12

Spacing of the spurs depends on the curvature of the bank being protected and is usually 1.5 to 2.0 times the length of the spurs. Spur dikes are susceptible to damage from floating ice and drift.

For protection against bottom scour and bank erosion, rip-rap can be placed along the dike. Stream flow and velocity will determine the maximum size and quantity of rip-rap required to protect each dike or series of dikes and rip-rap should be placed in accordance with Articles 3.6.4.2 through 3.6.4.4. In this application, rip-rap average size is not determined by wave height, but rather by the weight required, at the location in question, to avoid displacement during extreme flood conditions.

Installation on the larger waterways may require marine equipment which usually necessitates the services of a contractor; those on smaller waterways can often be built with land-based equipment. Untreated timber piles can be expected to have a service life of 10 years or more. If the installation is effective, the untreated piles will be covered with sediment by that time and no longer needed. If the installation is ineffective, replacement with some other type of construction, before the end of its service life, will be necessary. For this reason, it is usually not economical to use treated timber piles (References 5 and 9).

3.4.3.2 Crib Dikes

Crib dikes are frequently used where bedrock is exposed on the waterway bottom.

Crib dikes consist of log cribs approximately 10 feet square and filled with rock. Several cribs are cabled together to form a dike extending perpendicular out from the bank. In waterways carrying high velocity currents or large quantities of ice, dowels can be drilled in the rock bed to help maintain the dike's position.

Crib dikes serve much the same purpose as pile dikes except that no protective rip-rap is required because of the rock bottom.

3.4.3.3 Fence Jetties

Fence jetties are generally constructed with a series of units, normally cabled together and secured to the bank to insure action as a group.

Fence jetties are used to solve a variety of bank protection problems. They can be constructed parallel to the bank as well as transverse to the streamflow. Materials for construction can vary. Types often used are treated or untreated wood, used rails, pipe, and steel or concrete beams. The fencing material is usually wood or wire. If wire fencing is used, the required tensile strength depends on the design loading by the water and debris.

Fencing is not one of the most effective means of bank protection, but is commonly used because no special techniques for construction are required and materials are readily available.

Another way to use fences is to construct two fences parallel to the bank 3 to 10 feet apart. Brush, hay or rock is stacked between the fences, providing an extra measure of protection against the erosive action of the water currents. If the fences are constructed parallel to the bank and the bank is steep enough, a second fence is not required for holding the brush backfill. Fences constructed parallel to the bank generally serve as an erosion stopgap measure to allow sufficient time for the establishment of vegetation or to prevent sloughing of the bank. Fences constructed across part of the stream section promote sediment deposition. A transverse fence can be positioned to deflect debris downstream or to trap it. By constructing the fence so that it is oriented downstream at an oblique angle to the current flow, debris will be deflected into the main channel. This technique is useful if the stream has a heavy debris load and the designer desires to keep the banks clear. Conversely, the fence can be constructed so that it is oriented upstream at an oblique angle to the current flow. Debris is then trapped behind the fence. This construction method is effective for clearing the main channel of debris and serves to encourage sediment deposition.

Advantages:

1) Directs energy of river from bank to open channel.

2) Collects debris to slow current and promotes siltation.

Disadvantages:

1) Length is limited so not to interfere with navigation.

2) Susceptible to ice damage.

3) Costs are high.

4) Maintenance costs are high.

Maintenance Requirements:

Periodic repairs of damaged fence sections.

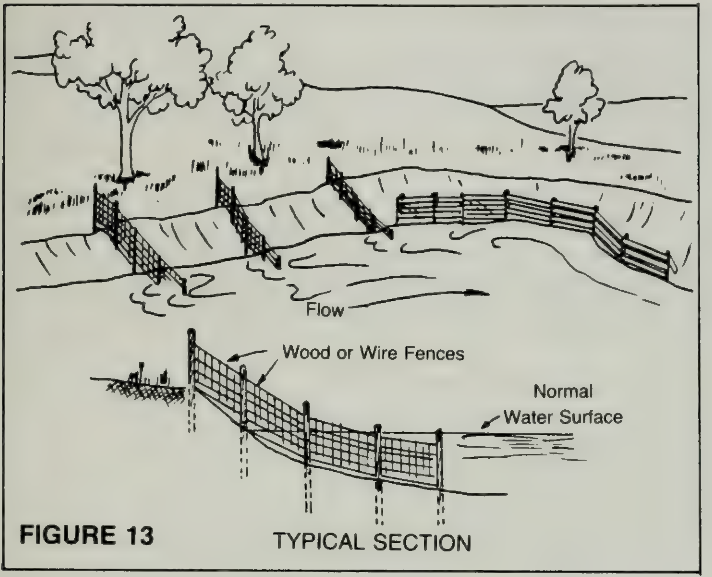


FIGURE 13

FENCE JETTIES

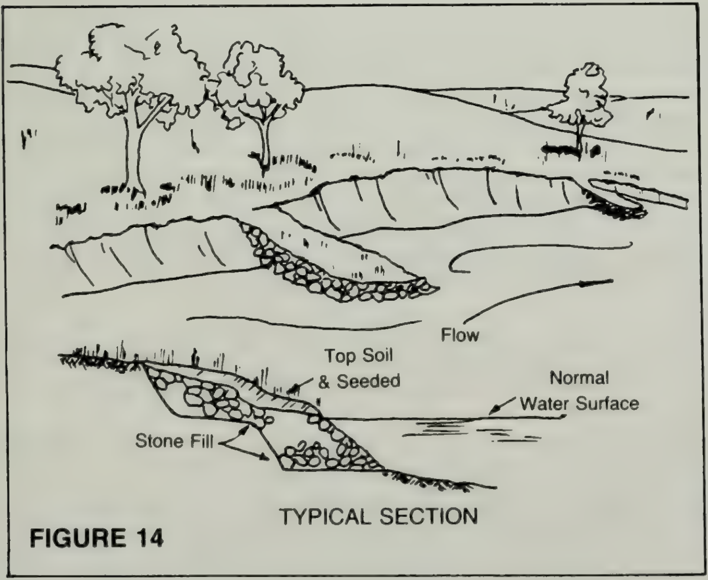


FIGURE 14

ROCK JETTY DIKES

3.4.3.4 Rock Jetty Dikes

Rock jetties are effective in deflecting the current away from the eroding bank. A series of jetties along the bank tends to create back currents between the jetties. This produces a siltation of foreshore between the projecting dikes of rock. Stone dikes are particularly effective protection for banks along streams carrying a minimum silt load. Stream velocity will determine the maximum size of stone required for satisfactory performance of a dike.

The rock jetty consists of stone-fill structures placed intermittently along an eroding bankline. The jetty is placed to protrude a short distance into a river channel near the normal water surface level. A root section should be constructed into the bankline to provide protection to prevent stream cutting action behind the hardpoint. For best results, a series of hardpoints can be used. Back currents are formed allowing siltation to occur. This will provide added bank protection.

Advantages:

1) No requirement for sediment deposit in hardpoint field.

2) Impermeability allows reduction of erosion force of stream.

3) Very little maintenance is required.

Disadvantages:

1) Heavy equipment required for construction.

2) Moderately high first cost.

3) Difficult to construct where access is limited.

Notes:

Construction materials are usually from quarry-run stone with specified limitations on the maximum size of stone and amount of fines. The dikes are built with crowns of various widths up to 10 feet or more depending on the severity of the expected attack, the method of construction, and the requirements for maintenance. The spacing between any two hardpoints has generally been related to the average of their lengths multipled by a spacing-length ratio. The spacing-length ratio is derived from the experience of the designer. The final selection of spacing and length may be an economic one, but for bank protection purposes, the length of the dikes should be just sufficient to move the eroding current away from the bank. Of course, there is a limitation on length since it must not be such that the dike will unduly restrict the navigation channel or increase the current velocity to an unacceptable value. Short rock jetties are effective in deflecting the current away from the eroding bank and tend to create a foreshore between the projecting dikes of rock. Use of broken concrete, if available, would eliminate some of the initial cost.

3.4.4 Gabions

Gabions are rectangular shaped, compartmented stone-filled baskets made from coated steel wire mesh and are used in waterway erosion control primarily as retaining structures, revetments (mattresses) and check dams.

Since gabions are flexible structures they can adjust to changes in the channel and thus can be used in a wide variety of situations. They are commercially available in several basket sizes.

The basket stone should be hard and durable and can be either quarry stone or river-bed cobbles. Stone sizes can range from 3 to 12 inches, depending on the application, with the minimum size being larger than the wire mesh openings. The use of clean stone in filling the gabions and exercising care in construction to prevent large voids, insures stability as well as permeability of the structure.

Construction of an adequate gabion base is essential in waterway use to reduce risk of scour of the foundation material and thus minimize the potential for undermining of the structure. Utilization of a geotextile fabric beneath a gabion revetment can improve performance of the system by impeding surface erosion. Some additional perspectives on gabion use can be found in References 1, 4, 9, 11 & 13.

3.4.5 Revetments

3.4.5.1 General Description

Revetments are used to line stream banks continuously, to prevent erosion, and protect the underlying bank. Construction of a revetment requires that the channel be lined with material capable of resisting the scouring velocity of the flow. A revetment is generally selected when limited channel right-of-way widths prohibit other less expensive types of construction. The revetment must be constructed so that it will not be undermined or out-flanked by continued bank erosion, as it would then become ineffective.

3.4.5.2 Typical Construction

The life of the revetment is largely dependent upon the manner in which the banks have been prepared to receive it. The bank should be graded to a slope generally not steeper than 2:1. A toe trench or cut-off wall should be considered. Suitable geotextile filter fabric should be placed under the revetment. The revetment should be constructed either of riprap properly sized or other types of suitable construction. Asphalt paving, articulated concrete slabs, or grout-filled mats have been successfully used in revetment construction. Proper sizing of the riprap to be used in a revetment requires analyzing the following factors: store shape, size, weight, gradation; channel side slopes, roughness, shape, alignment, invert slope; and riprap thickness layer. The Department of the Army, Corps of Engineers, has published a design manual, see Reference 6, which includes a detailed design for rip-rap protection.

Construction of a revetment is sometimes difficult in the channel. If conditions permit, it is sometimes easier to construct the revetment on dry land behind the channel bank and permit the erosion to continue up to the face of the newly constructed revetment.

3.4.6 Check Dams

Check dams are used to reduce erosion by reducing stream gradients in those streams where only small reduction in velocity will induce settlement of suspended silt. They can be used in side ditches, intercepting ditches, or where upstream approaches to bridges or culverts are very steep.

Check dams not only control stream bed erosion, but also serve to dissipate hydraulic energy at the vertical drop below each installation. They are made of various materials such as steel sheet piles, mass concrete, concrete or metal cribs, treated timber, gabions, rip-rap, or any other suitable materials. Appropriate protection measures must be considered to dissipate the increased flow velocity and prevent bed scouring downstream. An apron or other suitable structure is recommended to prevent undermining the check dam and in dissipating this energy.

3.4.7 Channel Cut-Offs

Channel cut-offs are feasible only when the potential property loss, exceeds or equals, access, right-of-way, legal and construction cost. If streamflow along the outside of a bend (the cut bank) is the principle reason, the relocation of the channel can be an economical alternative to more costly approaches to protect the bank.

Channel relocation is usually accomplished by cutting a pilot channel from a point upstream of the distressed bank to a point downstream from the bank. Because the slope of the pilot channel is steeper than the natural channel, the stream will start to flow through the pilot channel, thus increasing the water carrying capacity of the new channel. Eventually, the pilot channel will capture the streamflow as the channel enlarges and as the slower water movement through the natural channel is blocked by sediment deposition.

Another option to the natural blocking of the original channel by sediment would be to construct a closure dam across the upper end of the old channel to insure the diversion of the water and possibly speed up any repair of damage to the distressed bank.

Although excavating a pilot channel may provide a quick and relatively economical method for keeping streamflow away from an eroding bank, this action may cause serious future problems.

CAUTION: Shortening the length of a channel usually increases the gradient of the stream, thus resulting in higher stream velocity and a greater tendency to meander. As a consequence, bank erosion may result upstream from the pilot channel and flooding downstream. Bank protection appropriately placed can minimize these disadvantages.

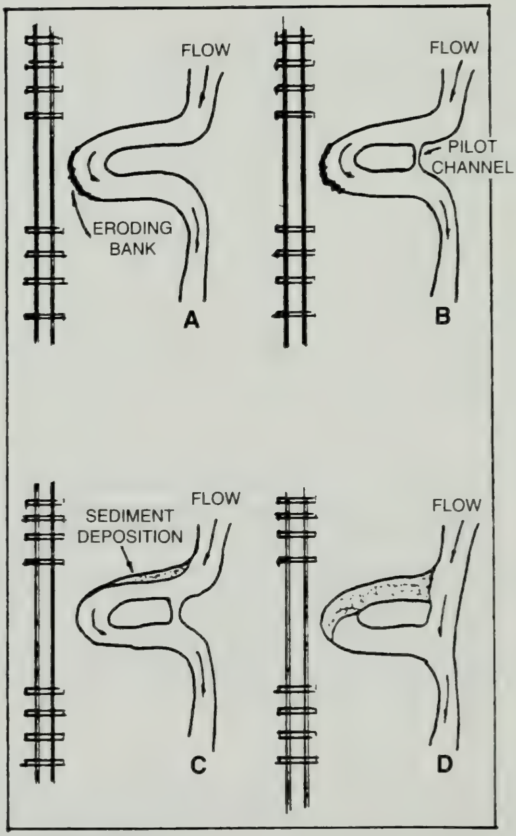


FIGURE 15

PROTECTION OF ERODING BANK BY REROUTING THE STREAM.

A. Eroding bank threatens railroad. B. Pilot channel is excavated. C. Pilot channel begins to capture streamflow; some sediment deposition occurs in the natural channel. D. Pilot channel has captured the streamflow; water movement through the natural channel is now blocked by sediment deposition and bank erosion is stopped. See "Caution" in text.

Bibliography

[1] "Streambank Protection Guidelines," U.S. Army Corps of Engineers, October, 1983.

[2] "Underwater Inspection and Repair of Bridge Substructures," Report No. 88, Synthesis of Highway Practice, National Cooperative Highway Research Program, Washington, D.C., 1981.

[3] "Scour at Bridge Waterways," Report No. 5, Synthesis of Highway Practice, National Cooperative Highway Research Program, Washington, D.C., 1970

[4] "Maccaferri Gabions and Reno Mattresses," Maccaferri Gabions, Inc., August, 1986.

[5] "Bank and Shore Protection in California Highway Practice," California Division of Highways, November, 1970

[6] "Hydraulic Design of Flood Control Channels," Department of the Army, Corps of Engineers, EM 1110-2-1601, July, 1970

[7] Proceedings, AREA, Vol. 56, p. 679

[8] Proceedings, AREA, Vol. 58, p. 717

[9] Proceedings, AREA, Vol. 60, p. 657

[10] Transactions, ASCE, Vol. 132, p. 211

[11] Transactions, ASCE, Vol. 134, p. 559

[12] Transactions, ASCE, Vol. 134, p. 878

[13] "Civil Engineering," March, 1974, Vol. 44, No. 3, P. 68

Proposed 1990 Manual Revisions To Chapter 2 — Track Measuring Systems

A definition for Track Quality Index (TQI) is proposed as follows:

1.2 Definition of Track Quality Index (TQI)—1989

TRACK QUALITY INDEX—A number, derived from a formula, used to represent observed track data, and which relates to the ability of the track to perform its function."

Proposed 1990 Manual Revisions To Chapter 5 — Track

Part 7—Rail Anchors has been rewritten to include additional specifications for remanufactured rail anchors and a section on "Recommended Practices for Rail Anchor Application and Maintenance" is to be added. Following are the new Part 7, Chapter 5 material:

AMERICAN RAILWAY ENGINEERING ASSOCIATION

Part 7

Rail Anchors

SPECIFICATIONS FOR RAIL ANCHORS

Revised 1990

Scope

These specifications define the requirements for drive on and spring type rail anchors, both new and remanufactured, which either may be applied manually or with standard anchor machines that readily can be adapted to the anchor being considered. (For the purpose of this specification, remanufactured anchors are defined as used drive on or spring type anchors which have been reformed to restore performance to the requirements described herein, except where specifically noted.)

Function

Rail anchors transmit static and dynamic longitudinal forces in the rail to the cross tie. A sufficient number of anchors are required to hold the rail in a fixed position.

Design

Rail anchors shall be designed:

(1) To function during their service life without damage to the rail base due to longitudinal forces, application forces, or vibratory action.

(2) To be able to resist longitudinal and torsional forces exerted by skewed ties without damage to the base of rail.

(3) To be capable of being applied tightly against the tie either manually or by machine.

(4) For easy application, removal and successive reapplication without appreciable loss of holding power as required by the following slip test.

Bearing Area

Rail anchors shall have sufficient bearing area and depth to minimize the possibility of the anchor damaging or becoming embedded in the tie under pressure and to prevent the anchor from overriding the tie.

Slip Test

Rail anchor slippage shall be determined by applying a gradually increasing load directly to the end of the rail; the anchor, applied perpendicular to the rail base, shall be resisted by a fixed metal block positioned 5/8th-inch below the base of the rail. In the interest of safety, the fixture should have clamps or other engagement which will prevent the rail and anchor from slipping off the fixed block while the load is being applied, but which will not restrain the rail from slipping through the anchor. The rail used in conducting the test shall meet AREA dimensional specification without any variation in the width and thickness of the base.

A. The rail shall be preloaded to 500 pounds to allow the anchor to take its initial set including anchor lean. The location of the anchor at the drive on or applied side of the rail base shall be marked or fixed by a dial gage. The load shall be increased at a rate not to exceed 1.0 inch per minute or 10,000 pounds per minute until it reaches 5,000 pounds where it shall be held for three minutes before measuring slippage, which shall not exceed a total of 1/16th-inch; the 5,000 pound load shall be held for an additional three minutes during which time there shall be no further slippage.*

Upon satisfactory completion of each test, the anchor being tested shall be removed, reapplied at a different location on the rail base, and shall meet the foregoing criteria for a second and a third successive test.

*NOTE: Some purchasers require that the load be applied to the end of the rail in increments varying from 500 to 1,000 pounds and held for 5 to 10 seconds to enhance uniformity of testing.

B. Slip tests shall be made as frequently as necessary by the manufacturer, *or as required by the purchaser,* during production to insure that the rail anchors meet the above criteria.

Fracture Test

New anchors from each production batch, or remanufactured anchors from each production batch of each anchor design submitted by the purchaser for remanufacturing shall be randomly selected and subject to fracture tests. The test shall be repeated three successive times on the same anchor with no fractures allowed.

For drive on anchors the test shall require a 275 foot-pound impact on the jaw of the anchor placed upright over a 30 degree angle wedge.

For spring type anchors the test shall require a force of sufficient magnitude to spread the anchor, without breaking, to a permanent dimension which exceeds the width of the rail base plus 0.05 inches, on which it is to be applied, by 6 percent. This measurement shall be made between points, in the same plane parallel to the rail base, where the anchor normally comes in contact with the rail base.

Retests and Rejection

Should a sample fail the fracture or slip test, a retest consisting of two anchors randomly selected from the same production lot shall be made. If both of these samples meet the test requirements, the anchors from that lot shall be accepted.

If either new anchor retest sample fails the fracture or slip test, all of the anchors in the production lot shall be rejected. Following rejection these anchors may be scrapped or reheat-treated at the manufacturer's discretion. Any reheat-treated anchors subsequently shall be tested using the original criteria.

If either remanufactured anchor retest sample fails the fracture or slip test, all of the anchors from the lot from which the samples were taken shall be rejected. Following rejection these anchors may be scrapped or reheat-treated at the manufacturer's discretion. Any reheat-treated anchors subsequently shall be tested using the original criteria.

Dimensions

The manufacturer shall carry out dimensional checks on 2 of every 500 anchors produced per line to assure compliance with all dimensional requirements. If either of these two anchors fail to conform, the manufacturer must take immediate remedial action to reject *all* nonconforming anchors. The foregoing dimensional checks will apply to remanufactured anchors only as related to those areas of the anchors which require reforming in order for the anchor to meet the requirements of this specification. The remanufacturer shall not be held responsible for the dimensional configuration or tolerances as related to original anchor design.

Workmanship

The finished rail anchors shall not be marred or deformed and shall be free of detrimental defects, such as, laps, cracks, seams and decarbonized or burned steel. It is recognized that remanufactured anchors may have defects that have resulted from previous service or application. Only those anchors having defects which may impair function that resulted from previous service or from the reforming process need be culled unless otherwise specified by the purchaser.

All anchors to be remanufactured shall be cleaned of oil, grease, or other residue so as to improve inspection and performance.

Identification

Each new rail anchor shall be hot-stamped to show the rail section and year of manufacture. Remanufactured anchors shall be identified as such by means of hot or cold stamping. The original manufacturer's identification as to rail section and year of manufacture will remain on the anchor.

Bagging

Unless otherwise specified all new or remanufactured anchors shall be bagged fifty (50) acceptable anchors per bag for shipment or storage. All anchors within each bag shall be for the same rail size and shall be of the same type of anchor. All correctly filled anchor bags shall have the open end of the bag closed completely and securely bound in such a manner as to insure that the bound end will not reopen during handling and shipping.

Tagging

Each bag shall have securely affixed to the outside an inspector's tag which shows the date of manufacture or remanufacture, type of anchor and the intended rail sizes covering the anchors contained in that bag.

Shipping Tag

Each load shall have securely affixed to not less than 10 percent of its bags tags which show consignee name and address, customer order number, manufacturer's order number, date of shipment and quantity of anchors in that shipment.

Inspection

When specified, the inspector representing the purchaser shall have free entry to the manufacturer's facilities and shall be provided, on a no charge basis, all reasonable facilities to satisfy him that the materials being furnished are in accordance with the purchaser's specifications. The plant manager shall be notified prior to the inspection so that the facilities, materials and product shall be available for inspection.

Acceptance

To be accepted, the rail anchors offered must fulfill all the requirements of these specifications.

RECOMMENDED PRACTICES FOR RAIL ANCHOR APPLICATION AND MAINTENANCE

Adopted 1990

1. Anchor shall be applied to the rail sections for which they are designed.
2. Rails having reduced base width and/or flange thickness account wear, corrosion, or rolling variations may require custom manufactured anchors for satisfactory performance. The purchaser or the anchor manufacturer's representative shall provide appropriate measurements of rail base to aid in optimizing rail anchor design.
3. Anchors applied at any one tie shall be of the same type.

4. Where the minimum recommended quantity of anchors is not sufficient to restrain the rail, additional anchors as determined by the Engineer/Maintenance shall be applied.

5. The base of the rail shall be reasonably clean and cribs open to accept the anchor and provide maximum rail restraint.

6. Whenever possible, anchors shall be applied against sound ties, preferably those with vertical sides. Anchors must be applied so as to have full bearing against the tie or tie plate. Before applying anchors which bear against the tie plate, the tie plate should be properly centered and spiked. Anchors should be applied from the gauge side of the rail when possible.

7. When rail or ties are renewed or respaced, anchors shall be properly repositioned.

8. Anchors shall be uniformly spaced along the rail length. To avoid tie skewing they must be installed on the same side of each tie on both rails.

9. Care shall be taken when applying rail anchors to ensure that they are properly seated. The manufacturer's instructions or recommendations must be followed.

10. Anchors must be applied and removed with the proper tools. Machines used to apply anchors must be properly adjusted, and adjustment must be checked daily or at more frequent intervals if necessary.

11. Anchors shall not be installed:

    (a) At ties which support rail joints.

    (b) Where they will interfere with bond wire, boot legs, insulated joints, or other signal or track appliances.

    (c) Within a minimum of two inches from the edge of any weld to prevent nicks or gouges within the heat affected zone of welds.

12. Rails shall be fully anchored immediately after laying. When necessary trains must be operated at slow speed until track is anchored at proper temperatures.

13. Sufficient anchors shall be applied and maintained through both tracks of turnouts,and on each side of turnouts to reduce rail movement. Additional anchors may be required to reduce rail movement at locations where gradients exist or where train braking or acceleration is common. Sufficient rail anchors shall be applied on all tracks approaching any crossings to prevent rail movement.

14. Damaged, defective, or ineffective anchors shall be replaced.

15. Anchors which become displaced shall be repositioned. Cause of rail movement shall be determined and additional anchors installed as necessary. When repositioning anchors by manual means they should be removed and applied rather than slid along the rail.

Proposed 1990 Manual Revisions To Chapter 7 - Timber Structures

In Part 2—Design of Wood Railway Bridges, it is proposed to revise Table 2.7.1 which contains allowable unit stresses for stress graded lumber. References to lumber grading rule agencies which are no longer active are deleted, and stress values in Table 2.7.1 reflect those currently used by the grading agencies. The proposed replacement pages for Table 2.7.1 follow.

Also, it is proposed to revise article 3.2.8.3 on Pile Covering to read:

3.2.8.3 Pile Covering

(a) The treated pile cut-off may be covered with plastic cement used with or without a fabric layer and topped with a 1/4" neoprene pad if desired.

(b) The use of roofing material or sheet metal to cover the cut-off has been found to retain moisture or increase wetting and is not recommended.

Table 2.7.1

ALLOWABLE UNIT STRESSES FOR STRESS GRADED LUMBER— RAILROAD LOADING (VISUAL GRADING)

		Dry Condition 19% Max. M.C.						Wet Condition Over 19% M.C.						Untreated Moderate Decay Hazard						
Species and Commercial Grade	Size Classification	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	Grading Rule Agency
EASTERN SPRUCE																				
Select Str		1075	725	65	350	825	1500	925	600	65	235	575	1450	825	550	65	235	525	1450	
No. 1		900	600	65	350	750	1500	775	500	65	235	525	1450	700	450	65	235	475	1450	NELMA
No. 2	2" to 4"	750	375	65	350	625	1400	650	325	65	235	450	1350	600	300	65	235	400	1350	NHPMA
No. 3	thick	425	225	65	350	400	1200	375	200	65	235	275	1150	350	175	65	235	250	1150	
Appearance	5" & wider	900	600	65	350	900	1500	775	500	65	235	625	1450	700	450	65	235	575	1450	
Stud		425	225	65	350	400	1200	375	200	65	235	275	1150	350	175	65	235	250	1150	
Select Str.	Beams & Strs.	950	650	60	350	675	1400	950	650	60	235	625	1400	850	575	60	235	575	1400	NELMA
No. 1		800	550	60	350	575	1400	800	550	60	235	525	1400	725	500	60	235	475	1400	NHPMA
Select Str.	Posts & Timber	900	600	60	350	700	1400	900	600	60	235	625	1400	800	550	60	235	575	1400	NELMA
No. 1		725	500	60	350	600	1400	725	500	60	235	550	1400	650	450	60	235	500	1400	NHPMA
																				NELMA
HEM—FIR																				
Select Str.		1250	850	70	365	1025	1500	1075	725	65	245	725	1450	975	650	65	245	650	1450	
No. 1	2" to 4"	1075	725	70	365	950	1500	925	600	65	245	675	1450	825	550	65	245	600	1450	
No. 2	thick	900	475	70	365	775	1400	775	400	65	245	550	1350	700	350	65	245	500	1350	WCLIB
No. 3	5" & wider	525	275	70	365	500	1200	450	230	65	245	350	1150	400	200	65	245	325	1150	WWPA
Appearance		1075	725	70	365	1125	1500	925	600	65	245	775	1450	825	550	65	245	700	1450	
Stud		525	275	70	365	500	1200	450	230	65	245	350	1150	400	200	65	245	325	1150	
Select Str.	Beams & Strs.	1175	675	65	365	825	1300	1175	675	65	245	750	1300	1050	600	65	245	675	1300	WCLIB
No. 1		950	475	65	365	675	1300	950	475	65	245	625	1300	850	425	65	245	550	1300	
Select Str.	Posts & Timber	1075	725	65	365	875	1300	1075	725	65	245	800	1300	975	650	65	245	725	1300	WCLIB
No. 1		875	575	65	365	775	1300	875	575	65	245	700	1300	775	525	65	245	625	1300	

Table 2.7.1 (Continued)

Species and Commercial Grade	Size Classification	Dry Condition 19% Max. M.C.						Wet Condition Over 19% M.C.						Untreated Moderate Decay Hazard						Grading Rule Agency
		F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	
Select Str.	Beams & Strs.	1125	650	65	365	825	1300	1125	650	65	245	750	1300	1000	575	65	245	675	1300	WWPA
No. 1		950	475	65	365	700	1300	950	475	65	245	625	1300	850	425	65	245	550	1300	
Select Str.	Posts & Timber	1075	725	65	365	875	1300	1075	725	65	245	800	1300	975	650	65	245	725	1300	WWPA
No. 1		850	575	65	365	775	1300	850	575	65	245	700	1300	775	525	65	245	625	1300	
DOUGLAS FIR SOUTH																				
Select Str.		1525	1025	80	470	1125	1400	1300	850	75	315	775	1350	1175	775	75	315	700	1350	
No. 1	2" to 4"	1300	875	80	470	1025	1400	1125	725	75	315	725	1350	1000	650	75	315	650	1350	
No. 2	thick	1075	550	80	470	850	1300	925	450	75	315	600	1250	825	400	75	315	550	1250	WWPA
No. 3	5" & wider	625	325	80	470	550	1100	525	275	75	315	375	1050	475	250	75	315	325	1050	
Appearance		1300	875	80	470	1225	1400	1125	725	75	315	850	1350	1000	650	75	315	775	1350	
Stud		625	325	80	470	550	1100	525	275	75	315	375	1050	475	250	75	315	325	1050	
Select Str.	Beams & Strs.	1400	950	75	470	900	1200	1400	950	75	315	825	1200	1250	850	75	315	750	1200	WWPA
No. 1		1175	775	75	470	775	1200	1175	775	75	315	700	1200	1050	700	75	315	625	1200	
Select Str.	Posts & Timber	1250	850	75	470	950	1200	1250	850	75	315	875	1200	1125	775	75	315	775	1200	WWPA
No. 1		1025	700	75	470	825	1200	1025	700	75	315	750	1200	925	625	75	315	675	1200	
DOUGLAS FIR-LARCH																				
Dense Select Str.		1900	1250	85	655	1475	1900	1625	1050	80	440	1025	1850	1450	950	80	440	925	1850	
Select Str.		1625	1075	85	560	1250	1800	1400	900	80	375	875	1750	1250	800	80	375	775	1750	
Dense No. 1	2" to 4"	1625	1075	85	655	1300	1900	1400	900	80	440	900	1850	1250	800	80	440	800	1850	WCLIB
No. 1	thick	1350	900	85	560	1125	1800	1150	750	80	375	775	1750	1025	675	80	375	700	1750	WWPA
Dense No. 2	5" & wider	1300	700	85	655	1125	1700	1125	600	80	440	775	1650	1000	550	80	440	700	1650	
No. 2		1125	575	85	560	950	1700	975	475	80	375	650	1650	875	425	80	375	575	1650	
No. 3		650	325	85	560	600	1500	550	275	80	375	425	1450	500	250	80	375	375	1450	
Appearance		1350	900	85	560	1350	1800	1150	750	80	375	950	1750	1025	675	80	375	850	1750	
Stud		650	325	85	560	600	1500	550	275	80	375	425	1450	500	250	80	375	375	1450	
Dense Select Str.		1700	1000	75	655	1175	1700	1700	1000	75	440	1075	1700	1525	900	75	440	975	1700	
Select Str.	Beams & Strs.	1450	850	75	560	1000	1600	1450	850	75	375	900	1600	1300	775	75	375	800	1600	WCLIB

Table 2.7.1 (Continued)

Species and Commercial Grade	Size Classification	Dry Condition 19% Max. M.C.						Wet Condition Over 19% M.C.						Untreated Moderate Decay Hazard						Grading Rule Agency
		F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	
Dense No. 1		1400	700	75	655	1000	1700	1400	700	75	440	900	1700	1250	625	75	440	800	1700	
No. 1		1175	600	75	560	825	1600	1175	600	75	375	750	1600	1050	550	75	375	675	1600	
Dense Select Str.		1575	1025	75	655	1225	1700	1575	1025	75	440	1125	1700	1425	925	75	440	1000	1700	
Select Str.	Posts & Timber	1350	900	75	560	1025	1600	1350	900	75	375	925	1600	1225	800	75	375	825	1600	WCLIB
Dense No. 1		1250	850	75	655	1075	1700	1250	850	75	440	975	1700	1125	775	75	440	875	1700	WWPA
No. 1		1075	750	75	560	900	1600	1075	750	75	375	825	1600	975	675	75	375	750	1600	
Dense Select Str.		1700	1125	75	655	1175	1700	1700	1125	75	440	1075	1700	1525	1000	75	440	975	1700	
Select Str.		1450	950	75	560	1000	1600	1450	950	75	375	900	1600	1300	850	75	375	800	1600	WWPA
Dense No. 1		1400	950	75	655	1000	1700	1400	950	75	440	900	1700	1250	850	75	440	800	1700	
No. 1		1225	800	75	560	825	1600	1225	800	75	375	750	1600	1100	725	75	375	675	1600	
SOUTHERN PINE																				
Select Str.		1575	1025	80	510	1225	1700	1250	800	75	340	800	1500	1125	725	75	340	725	1500	
Dense Select Str.		1850	1175	80	595	1450	1800	1450	950	75	400	950	1600	1300	850	75	400	850	1600	
No. 1	2" to 4"	1300	875	80	510	1125	1700	1075	700	75	340	750	1500	975	625	75	340	675	1500	
No. 1 Dense	thick	1525	1025	80	595	1300	1800	1250	825	75	400	850	1600	1125	750	75	400	775	1600	SPIB
No. 2	5" & wider	1075	550	80	510	900	1600	875	450	75	340	600	1400	775	400	75	340	550	1400	
No. 2 Dense		1250	650	80	595	1075	1600	1025	550	75	400	725	1400	925	500	75	400	650	1400	
No. 3		625	325	80	510	550	1400	500	275	75	340	375	1200	450	250	75	340	325	1200	
No. 3 Dense		750	375	80	595	650	1500	575	325	75	400	425	1300	525	275	75	400	375	1300	
Stud		650	325	80	510	550	1400	525	275	75	340	375	1200	475	250	75	340	325	1200	
Dense Str. 86	2" to 4"	2350	1575	140	595	1800	1800	—	—	—	—	—	—	—	—	—	—	—	—	
Dense Str. 72	thick	1975	1300	115	595	1475	1800	—	—	—	—	—	—	—	—	—	—	—	—	SPIB
Dense Str. 65		1800	1175	105	595	1350	1800	—	—	—	—	—	—	—	—	—	—	—	—	
Dense Str. 86	2½" and	—	—	—	—	—	—	1900	1250	130	395	1175	1600	1700	1125	130	395	1050	1600	
Dense Str. 72	thicker	—	—	—	—	—	—	1575	1075	110	395	1000	1600	1425	975	110	395	900	1600	SPIB
Dense Str. 65		—	—	—	—	—	—	1450	950	100	395	900	1600	1300	850	100	395	800	1600	

Table 2.7.1 (Continued)

Species and Commercial Grade	Size Classification	Dry Condition 19% Max. M.C.						Wet Condition Over 19% M.C.						Untreated Moderate Decay Hazard						Grading Rule Agency
		F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	F_b psi	F_t psi	F_v psi	$F_{c\perp}$ psi	F_c psi	E ksi	
No. 1 SR		—	—	—	—	—	—	**1225**	**775**	**100**	340	**700**	**1500**	**1100**	**700**	**100**	340	**625**	**1500**	
No. 1 Dense SR	**5" and**	—	—	—	—	—	—	**1400**	**950**	**100**	395	**825**	**1600**	**1250**	**850**	**100**	395	**750**	**1600**	**SPIB**
No. 2 SR	**thicker**	—	—	—	—	—	—	**1000**	**650**	**85**	340	**550**	**1400**	**900**	**575**	**85**	340	**500**	**1400**	
No. 2 Dense SR		—	—	—	—	—	—	**1125**	**775**	**85**	395	**650**	**1400**	**1000**	**700**	**85**	395	**575**	**1400**	
MIXED MAPLE																				
Select Str.		1175	800	90	560	800	1300	1000	675	85	375	550	1250	900	610	85	375	500	1250	
No. 1		1000	675	90	560	725	1300	850	575	85	375	500	1250	775	525	85	375	450	1250	
No. 2	2" to 4"	825	425	90	560	600	1100	700	350	85	375	425	1050	625	325	85	375	375	1050	NELMA
No. 3	thick	500	250	90	560	375	1000	425	200	85	375	275	950	375	375	85	375	250	950	
Appearance	5" & wider	1000	675	90	560	875	1300	850	575	85	375	625	1250	775	775	85	375	575	1250	
Stud		475	250	90	560	375	1000	400	200	85	375	275	950	350	350	85	375	250	950	
Select Str.	Beams & Strs.	1025	625	80	560	650	1100	1025	625	80	375	600	1100	925	925	80	375	550	1100	NELMA
No. 1		875	450	80	560	550	1100	875	450	80	375	500	1100	800	800	80	375	450	1100	
Select Str.	Posts & Timber	1000	650	80	560	675	1100	1000	650	80	375	625	1100	900	900	80	375	575	1100	NELMA
No. 1		800	550	80	560	575	1100	800	550	80	375	525	1100	725	725	80	375	475	1100	
RED OAK																				
Select Str.		1350	900	75	740	950	1400	1150	750	75	495	675	1350	1025	675	75	495	600	1350	
No. 1	2" to 4"	1175	775	75	740	825	1400	1000	650	75	495	575	1350	900	575	75	495	525	1350	
No. 2	thick	950	500	75	740	700	1200	825	425	75	495	500	1150	750	375	75	495	450	1150	NELMA
No. 3	5" & wider	575	300	75	740	450	1100	500	250	75	495	325	1050	450	225	75	495	300	1050	
Appearance		1175	775	75	740	1000	1400	1000	650	75	495	700	1350	900	575	75	495	625	1350	
Stud		575	300	75	740	450	1100	500	250	75	495	325	1050	450	225	75	495	300	1050	
Select Str.	Beams & Strs.	1200	725	70	740	750	1200	1200	725	70	495	675	1200	1075	650	70	495	600	1200	NELMA
No. 1		1025	500	70	740	625	1200	1025	500	70	495	575	1200	925	450	70	495	525	1200	
Select Str.	Posts & Timber	1125	775	70	740	800	1200	1125	775	70	495	550	1200	1025	700	70	495	500	1200	NELMA
No. 1		900	600	70	740	700	1200	900	600	70	495	500	1200	800	550	70	495	450	1200	

Table 2.7.1 (Continued)

1 Abbreviations used in this table are as follows:

NELMA—Northeastern Lumber Manufacturers Association, Inc.
NHPMA—Northern Hardwood & Pine Manufacturers Association, Inc.
SPIB—Southern Pine Inspection Bureau
WCLIB—West Coast Lumber Inspection Bureau
WWPA—Western Wood Products Association
F_b—Unit Stress in Extreme Fiber in Bending
F_t—Unit Stress in Tension Parallel to the Grain
F_v—Unit Stress in Horizontal Shear
$F_{c\perp}$—Unit Stress in Compression Perpendicular to the Grain
F_c—Unit Stress in Compression Parallel to the Grain
E—Modulus of Elasticity
Str.—Structural
Strs—Stringers
M C—Moisture Content

2 Dry condition design values apply to lumber that will be used under dry conditions, such as in most covered structures. For 2" to 4" thick lumber, the DRY surfaced size shall be used. In calculating design values, the natural gain in strength and stiffness that occurs as lumber dries has been taken into consideration, as well as the reduction in size that occurs when unseasoned lumber shrinks. The gain in load carrying capacity, due to increased strength and stiffness resulting from drying, more than offsets the design effect of size reductions due to shrinkage. For 5" and thicker lumber, the surfaced sizes also may be used because design values have been adjusted to compensate for any loss in size by shrinkage which may occur.

3. Untreated Lumber Moderate Decay Hazard is the hazard that exists in ordinary construction, not in contact with the ground, exposed to the weather in most parts of the United States. Where a greater hazard exists, make further reductions in working unit stresses. For lumber treated with preservative, it is common practice not to reduce allowable stresses because of decay hazard.

4. Tabulated tension parallel to grain for all species 5" and wider, 2" to 4" thick size classifications apply only to 5" and 6" widths for grades of Select Structural No. 1, No. 2, No. 3, Appearance and Stud (including dense grades).

For lumber wider than 6" in these grades, the tabulated F_t values shall be multiplied by the following factors:

Grade 2" to 4" thick, 5" and wider (includes dense grades)	Multiply tabulated F_t values by 5" & 6" wide	 8" wide	 10" & wider
Select Structural	1.00	0.90	0.80
No. 1, No. 2, No. 3 & Appearance	1.00	0.80	0.60
Stud	1.00	—	—

5. Design values for all species of stud grade in 5" and wider size classifications apply to 5" and 6" widths only.

6. The values for dimension lumber 2" to 4" in thickness are based on edge-wise use. When such lumber is used flat-wise, the design values for extreme fiber in bending for all species may be multiplied by the following factors:

Dimension Lumber used Flat-wise

Width	*Thickness* 2"	 3"	 4"
2" to 4"	1.10	1.04	1.00
5" and wider	1.22	1.16	1.11

7. The tabulated horizontal shear values shown herein for lumber 4" and thinner, except Southern Pine, should be multiplied by 0.92 when such lumber is manufactured unseasoned. Specific horizontal shear values may be established by use of the following table when the length of split or checks is known.
(Not applicable to Southern Pine.)

Shear Stress Modification Factors

When Length of Split on Wide Face is:	Multiply Tabulated F_v Value by (Nominal 2" Lumber)	When Length of Split on Wide Face is:	Multiply Tabulated F_v (3" & Thicker Lumber)
No split	2.00	No split	2.00
1/2 x wide face	1.67	1/2 x narrow face	1.67
3/4 x wide face	1.50	1 x narrow face	1.33
1 x wide face	1.33	1-1/2 x narrow face or more	1.00
1-1/2 x wide face or more	1.00		

8. Timbers are intended for the use of the whole piece on one span. If the piece is to be used continuous over two spans, the limitations prescribed for the middle one-third and the middle one-half should be applied to the middle two-thirds and middle three-quarters, respectively. If the piece is to be used continuous over three or more spans, the limitations prescribed for the middle one-third and middle one-half should be applied throughout the length of the piece.

9. For normal conditions other than railroad loading allowable unit stresses may be multiplied by 1.11 for F_b, F_t, F_v, $F_{c\perp}$, and F_c, E shall remain unchanged.

Proposed 1990 Manual Revisions To Chapter 8 — Concrete Structures and Foundations

Various changes are proposed to articles in Part 2—Reinforced Concrete Design and are detailed below. Part 4—Pile Foundations has been rewritten with some of the salient revisions to articles being:

Article 4.1.1 (b)—Addition of pile definition section.

Article 4.2.1.1—Live load forces are broken into vertical and lateral cases to insure consideration of the combination of their effects in design. Defined when live load impact may be applied.

Article 4.2.2—Complete revision of section on loads on pile to better define the pile actions to be investigated and what safety factors are to be used.

Article 4.2.3—Revision of the section on eccentricity of piles to focus the minimization of eccentric loads on pile groups to primary forces.

Article 4.2.4.—Revision of factors of safety to be used when considering pile uplift.

Article 4.2.6—Revision of the batter pile section to require all primary lateral forces to be taken by batter piles.

Article 4.3.5—Revised to allow the use of previous pile driving records, under certain circumstances, to estimate pile lengths.

Article 4.5.1.1—Revised to limit the minimum weight of the striking parts of the hammer to one-fourth the weight of the pile for precast concrete piles.

The new Part 4 is shown on pages 74 to 86.

Part 2 — Reinforced Concrete Design

The following changes proposed for articles in Part 2 are:

Article 2.1.5 (Page 8-2-2)

Replace existing article with the following:

2.1.5 Pier Protection

2.1.5.1 Adjacent to Railroad Tracks

To limit damage by the redirection and deflection of railroad equipment, piers supporting bridges over railways and with a clear distance of less than 25 feet from the centerline of a railroad track shall be of heavy construction (defined below) or shall be protected by a reinforced concrete crash wall. Crash walls for piers from 12 to 25 feet clear from the centerline of track shall have a minimum height of 6 feet above the top of rail. Piers less than 12 feet clear from the centerline of track shall have a minimum crash wall height of 12 feet above the top of rail.

The crash wall shall be at least 2 feet, 6 inches thick and at least 12 feet long. When two or more columns compose a pier, the crash wall shall connect the columns and extend at least 1 foot beyond the outermost columns parallel to the track. The crash wall shall be anchored to the footings and columns, if applicable, with adequate reinforcing steel and shall extend to at least 4 feet below the lowest surrounding grade.

Piers shall be considered of heavy construction if they have a cross-sectional area equal to or greater than that required for the crash wall and the larger of its dimensions is parallel to the track.

Consideration may be given to providing protection for bridge piers over 25 feet from the centerline of track as conditions warrant. In making this determination, account shall be taken of such factors as horizontal and vertical alignment of the track, embankment height, and an assessment of the consequences of serious damage in the case of a collision.

2.1.5.2 Over Navigable Streams

Piers located adjacent to channels of navigable waterways shall have a protection system in accordance with Part 23, this chapter.

Article 2.2.1 (Pages 8-2-2 and 8-2-4)

Add the following new notations:

a_v = shear span, distance between concentrated load and face of support—Articles 2.29.7 and 2.35.7.

A_f = area of reinforcement in bracket or corbel resisting moment, sq. in.—Articles 2.29.7 and 2.35.7.

A_h = area of shear reinforcement parallel to flexural tension reinforcement, sq. in.—Articles 2.29.7 and 2.35.7

A_n = area of reinforcement in bracket or corbel resisting tensile force, $N_c(N_{uc})$, sq. in.—Articles 2.29.7 and 2.35.7.

N_{uc} = factored tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension—Article 2.35.7

Article 2.2.1 (Page 8-2-3)

Add the following new notation:

α_f = angle between shear-friction reinforcement shear plane—Article 2.29.4 and 2.35.4
and
λ = correction factor related to unit weight of concrete—Articles 2.29.4 and 2.35.4

Article 2.2.1 (Pages 8-2-3, 8-2-4 and 8-2-5)

Delete notation f_h, ℓ_e, and ζ and add notations ℓ_{dh} and ℓ_{hb} to read as follows:

ℓ_{dh} = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus radius of bend and one bar diameter), in.

= ℓ_{hb} x applicable modification factors

ℓ_{hb} = basic development length of standard hook in tension, in.

Article 2.2.1 (Page 8-2-4)

Add the following new notation:

N_c = Design tensile force applied at top of bracket or corbel acting simultaneously with V, to be taken as positive for tension.—Article 2.29.7

Article 2.2.1 (Pages 8-2-4 and 8-2-5)

Delete existing notations M_1 M_2, and δ, and add the following new notations:

M_{1b} = Value of smaller end moment on compression member due to loads that result in no appreciable sidesway, calculated by conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature.—Article 2.34.2.

M_{2b} = Value of larger end moment on compression member due to loads that result in no appreciable sidesway, calculated by conventional elastic frame analysis, always positive.—Article 2.34.2

M_{2s} = Value of larger end moment on compression member due to loads that result in appreciable sidesway, calculated by conventional elastic frame analysis, always positive.—Article 2.34.2

δ_b = Moment magnification factor for members braced against sidesway to reflect effects of member curvature between ends of compression member.

δ_s = Moment magnification factor for members not braced against sideway to reflect lateral drift resulting from lateral and gravity loads.

Article 2.4.1 (Page 8-2-12)

Revise article 2.4.1 to read as follows:

2.4.1 Standard hooks

The term "standard hook" as used herein, shall mean one of the following:

(a) 180-deg bend plus $4d_b$ extension, but not less than 2-1/2 in. at free end of bar.

(b) 90-deg bend plus $12d_b$ extension at free end of bar.

(c) For stirrup and tie hooks
  (1) #5 bar and smaller, 90-deg bend plus $6d_b$ extension at free end of bar, or
  (2) #6, #7, and #8 bar, 90-deg bend plus $12d_b$ extension at free end of bar, or
  (3) #8 bar and smaller, 135-deg bend plus $6d_b$ extension at free end of bar.

Article 2.4.2 (Page 8-2-12)

Revise paragraph (a) to read as follows:

(a) Diameter of bend measured on the inside of the bar, other than for stirrups and ties in sizes # 3 through #5, shall not be less than the values in Table 2.4.

Article 2.4.2 (Page 8-2-12)

Revise paragraph (b) to read as follows:

(b) Inside diameter of bends for stirrups and ties shall not be less than $4d_b$ for #5 bar and smaller. For bars larger than #5, diameter of bend shall be in accordance with Table 2.4.

Article 2.13.2 (Page 8-2-18)

Revise paragraph (c) in part to read as follows:

(c) At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that ℓ_d computed for f_y by Article 2.14 satisfies Eq. (2-4); except Eq. (2-4) need not be satisfied for reinforcement terminating beyond centerline of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$d \leq \frac{M}{V} + \ell_a \qquad (2\text{-}4)$$

where M is the computed moment capacity assuming all positive moment tension reinforcement at the section to be fully stressed. V is the maximum applied design shear at the section. ℓ_a shall be the embedment length beyond center of support or point of inflection. ℓ_a at a point of inflection shall be limited to the effective depth of the member or $12d_v$, whichever is greater. The value M/V in the development length limitation may be increased 30 percent when the ends of the reinforcement are confined by a compressive reaction.

Article 2.17 (page 8-2-20)

Revise article 2.17 (in total) to read as follows:

2.17 Development of standard hooks in tension

(a) Development length ℓ_{dh}, in inches, for deformed bars in tension terminating in a standard hook (Article 2.4.1) shall be computed as the product of the basic development length ℓ_{hb} of Paragraph (b) and the applicable modification factor or factors of Paragraph (c), but ℓ_{dh} shall not be less than $8d_b$ or 6 in., whichever is greater.

(b) Basic development length ℓ_{hb} for a hooked bar with f_y equal to 60,000 psi shall be .... $1200d_b/\sqrt{f'_c}$.

(c) Basic development length ℓ_{hb} shall be multiplied by applicable modification factor or factors for:

(1) Bar yield strength
Bars with f_y other than 60,000 psi .... $f_y/60{,}000$.

(2) Concrete cover
For #11 bar and smaller, side cover (normal to plane of hook) not less than 2-1/2 in., and for 90 deg hook, cover on bar extension beyond hook not less than 2 in. 0.7

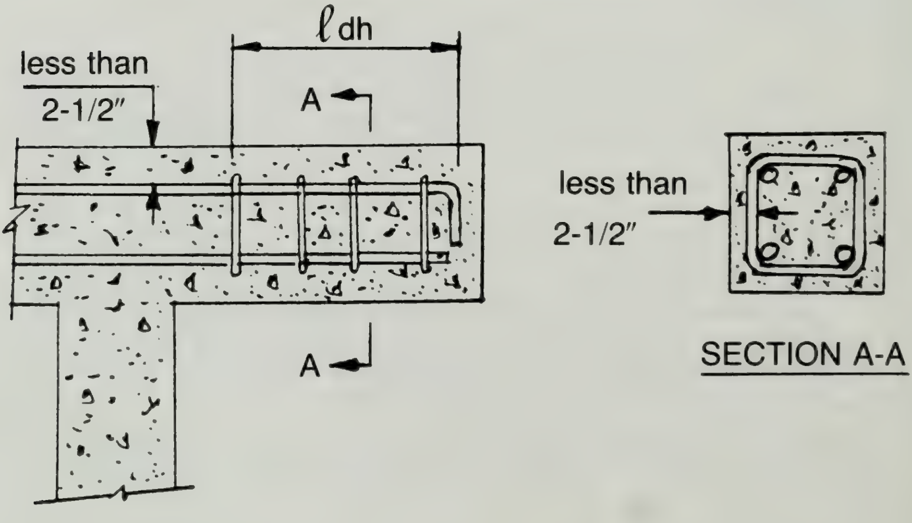
(3) Ties or stirrups
For #11 bar and smaller, hook enclosed vertically or horizontally within ties or stirrup-ties spaced along full development length ℓ_{dh} not greater than $3d_b$, where d_b is diameter of hooked bar 0.8

(4) Excess reinforcement
Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis .... (A_s required)/(A_s provided).

(5) Lightweight aggregate concrete 1.3

(d) For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 2-1/2 in., hooked bar shall be enclosed within ties or stirrups spaced along full development length ℓ_{dh} not greater than $3d_b$, where d_b is diameter of hooked bar. For this case, factor of Paragraph (c)(3) shall not apply.

(e) Hooks shall not be considered effective in developing bars in compression.



Article 2.18 and 2.20 (Pages 8-2-20 & 8-2-21)

Revise artricle 2.20 to read as follows; delete existing Article 2.18:

2.20 Mechanical Anchorage

(a) Any mechanical device shown by tests to be capable of developing the strength of reinforcement without damage to concrete may be used as anchorage.

(b) Develoment of reinforcement may consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between point of maximum bar stress and the mechanical anchorage.

Article 2.22.3 (Page 8-2-23)

In paragraph (e) change amount of stagger in adjacent spliced bars from "ℓ_d" to "30 in."

Article 2.29.1 and 2.35.1 (Pages 8-2-30 and 8-2-41)

Add the following sentence to paragraph (b) of articles 2.29.1 and 2.35.1

"An exception occurs when major concentrated loads are imposed between that point and the face of support. In that case sections closer than d to the support shall be designed for V_u at distance d plus the major concentrated loads."

Article 2.29.2 (Page 8-2-31)

Add a new paragraph (e) to read as follows; and renumber existing (e) to (f):

(e) Special provisions for slabs of box culverts. For slabs of box culverts under 2 feet or more fill, shear stress v_c may be computed by

$$v_c = \sqrt{f'_c} + 2200\rho \frac{Vd}{M} \qquad (2\text{-}10)$$

but v_c shall not exceed $1.8\sqrt{f'_c}$. For single cell box culverts only, v_c need not be taken less than $1.4\sqrt{f'_c}$ for slabs monolithic with walls or $1.2\sqrt{f'_c}$ for slabs simply supported. The quantity of Vd/M shall not be taken greater than 1.0, where M is moment occurring simultaneously with V at section considered.

Article 2.29.4 (Page 8-2-32)

Revise article 2.29.4 to read as follows:

2.29.4 Shear-friction

(a) Provisions for shear-friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

(b) A crack shall be assumed to occur along the shear plane considered. Required area of shear-friction reinforcement A_{vf} across the shear plane may be designed using either paragraph (c) or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests. Provisions of paragraph (d) through (h) shall apply for all calculations of shear transfer strength.

(c) Shear-friction design method

(1) Shear-friction reinforcement is perpendicular to shear plane, area of shear-friction reinforcement A_{vf} shall be computed by

$$A_{vf} = \frac{V}{f_s \mu} \qquad (2\text{-}13)$$

where μ is the coefficient of friction in accordance with paragraph (c) (3).

(2) When shear-friction reinforcement is inclined to shear plane such that the shear force produces tension in shear-friction reinforcement, area of shear-friction reinforcement A_{vf} shall be computed by

$$A_{vf} = \frac{V}{f_s\,(\mu \sin \alpha_f + \cos \alpha_f)} \qquad (2\text{-}14)$$

where α_f is angle between shear-friction reinforcement and shear plane.

(3) Coefficient of friction μ in Eq. (2-13) and Eq. (2-14) shall be

concrete placed monolithically 1.4λ

concrete placed against hardened concrete with surface intentionally roughened as specified in paragraph (g) 1.0λ

concrete placed against hardened concrete not intentionally roughened 0.6λ

concrete anchored to as-rolled structural steel by headed studs or by λ reinforcing bars (see paragraph h) 0.7λ

where λ = 1.0 for normal weight concrete; 0.85 for "sand-lightweight" concrete; and 0.75 for "all lightweight" concrete. Linear interpolation may be applied when partial sand replacement is used.

(d) Shear stress v on area of concrete section resisting shear transfer shall not exceed $.09f'_c$ nor 360 psi.

(e) Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane may be taken as additive to the force in the shear-friction reinforcment $A_{vf}f_s$, when calculating required A_{vf}.

(f) Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

(g) For the purpose of Article 2.29.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, interface shall be roughened to a full amplitude of approximately 1/4 in.

(h) When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

Article 2.29.5 (Pages 8-2-32 and 8-2-33)

Revise article 2.29.5 to read as follows:

2.29.5 Horizontal shear design for composite concrete flexural members.

(a) In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

(b) Design of cross sections subject to horizontal shear may be in accordance with provisions of Paragraph (c) or (d), or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests.

(c) Design horizontal shear stress v_{dh} at any cross section may be computed by

$$v_{dh} = \frac{V}{b_v d} \qquad (2\text{-}14)$$

where V is design shear force at section considered and d is for entire composite section.

Horizontal shear v_{dh} shall not exceed permissible horizontal shear v_h in accordance with the following:

(1) When contact surface is clean, free of laitance, and intentionally roughened, shear stress v_h shall not exceed 36 psi.

(2) When minimum ties are provided in accordance with paragraph (e), and contact surface is clean and free of laitance, but not intentionally roughened, shear stress v_h shall not exceed 36 psi.

(3) When minimum ties are provided in accordance with paragraph (e), and contact surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in., shear stress v_h shall not exceed 160 psi.

(4) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by paragraph (e), permissible v_h may be increased by $72f_y/40{,}000$ psi.

(d) Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the actual change in compressive or tensile force, and provisions made to transfer that force as horizontal shear between interconnected elements. Horizontal shear shall not exceed the permissible horizontal shear stress v_h in accordance with paragraph (c).

(e) Ties for horizontal shear

(1) A minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50b_v s/f_y$, and tie spacing 's' shall not exceed 4 times the least web width of support element, nor 24 in.

(2) Ties for horizontal shear may consist of singles bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed). All ties shall be adequately anchored into interconnected elements by embedment or hooks.

Article 2.34.2 (Page 8-2-39)

Revise the part of paragraph (e) on page 8-2-39 to read as follows:

(e) Compression members shall be designed using the factored axial load P_u from a conventional frame analysis and a magnified factored moment M_c defined by Eq. (2-32). For members braced against sidesway, δ_s shall be taken as 1.0. For members not braced against sidesway, δ_b shall be evaluated as for a braced member and δ_s as for an unbraced member.

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \qquad (2\text{-}32)$$

where

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} \geq 1.0 \qquad (2\text{-}33)$$

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_u}{\phi \Sigma P_c}} \geq 1.0 \qquad (2\text{-}34)$$

and

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \qquad (2\text{-}35)$$

Article 2.34.2 (Page 8-2-40)

Add a new paragraph (f) to read as follows and delete existing paragraph (i). Renumber existing paragraphs (f), (g), and (h) to (g), (h), and (i) respectively.

(f) When a group of compression members on one level composes a bent, or when they are connected integrally to the same superstructure, and all collectively resist the sidesway of the structure, the value of δ_s shall be computed for the member group with ΣP_u and ΣP_c equal to the summations for all compression members in the group.

Article 2.34.2 (Pages 8-2-39 and 8-2-40)

Throughout Article 2.34.2 change all notation "M_1" to "M_{1b}", and "M_2" to "M_{2b}".

Article 2.35.2 (Page 8-2-41)

Add a new paragraph (e) to read as follows, and renumber existing (e) to (f):

(e) Special provisions for slabs of box culverts. For slabs of box culverts under 2 feet or more fill, shear stress v_c may be computed by

$$v_c = 2.14 \sqrt{f'_c} + 4600 \rho \frac{V_u d}{M_u} \qquad (2\text{-}42)$$

but v_c shall not exceed $4\sqrt{f'_c}$. For single cell box culverts only, v_c need not be taken less than $3\sqrt{f'_c}$ for slabs monolithic with walls or $2.5\sqrt{f'_c}$ for slabs simply supported. The quantity $V_u d/M_u$ shall not be taken greater than 1.0, where M_u is factored moment occurring simultaneously with V_u at section considered.

Article 2.35.4 (Page 8-2-42)

Revise article 2.35.4 to read as follows:

2.35.4 Shear-friction

(a) Provisions for shear-friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

(b) A crack shall be assumed to occur along the shear plane considered. Required area of shear-friction reinforcement A_{vf} across the shear plane may be designed using either paragraph (c) or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests. Provisions of paragraph (d) through (h) shall apply for all calculations of shear transfer strength.

(c) Shear-friction design method

(1) When shear-friction reinforcement is perpendicular to shear plane, area of shear-friction reinforcement A_{vf} shall be computed by

$$A_{vf} = \frac{V_u}{\phi f_y \mu} \qquad (2\text{-}45)$$

where μ is the coefficient of friction in accordance with paragraph (c) (3).

(2) When shear-friction reinforcement is inclined to shear plane such that the shear force produces tension in shear-friction reinforcement, area of shear friction reinforcement A_{vf} shall be computed by

$$A_{vf} = \frac{V_u}{\phi f_y (\mu \sin \alpha_f + \cos \alpha_f)} \qquad (2\text{-}46)$$

where α_f is angle between shear-friction reinforcement and shear plane.

(3) Coefficient of friction μ in Eq. (2-45) and Eq. (2-46) shall be

concrete placed monolithically 1.4λ

concrete placed against hardened concrete with surface intentionally roughened as specified in paragraph (g) 1.0λ

concrete placed against hardened concrete not intentionally roughened 0.6λ

concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see paragraph 'h') 0.7λ

where $\lambda = 1.0$ for normal weight concrete; 0.85 for "sand-lightweight" concrete; and 0.75 for "all lightweight" concrete. Linear interpolation may be applied when partial sand replacement is used.

(d) Shear stress v_u on area of concrete section resisting shear transfer shall not exceed $0.2f'_c$ nor 800 psi.

(e) Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vf}f_y$, when calculating required A_{vf}.

(f) Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

(g) For the purpose of Article 2.35.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, interface shall be roughened to a full amplitude of approximately 1/4 in.

(h) When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

Article 2.35.7 (Page 8-2-44)

Add a new article 2.35.7 to read as follows:

2.35.7 Special provisions for brackets and corbels

(a) Provisions of Article 2.35.7 shall apply to brackets and corbels with a shear span-to-depth ratio and a $_v$/d not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u. Distance d shall be measured at face of support.

(b) Depth at outside edge of bearing area shall not be less than 0.5d.

(c) Section at face of support shall be designed to resist simultaneously a shear V_u, a moment $[V_ua_v + N_{uc}(h-d)]$, and a horizontal tensile force N_{uc}.

(1) In all design calculations in accordance with Article 2.35.7, strength reduction factor ϕ shall be taken equal to 0.85.

(2) Design of shear-friction reinforcement A_{vf} to resist shear V_u shall be in accordance with Article 2.35.4. For normal weight concrete, shear stress v_u shall not exceed $0.2f'_c$ nor 800 psi. For "all lightweight" or "sand-lightweight" concrete, shear stress v_u shall not exceed $(0.2 - 0.07a_v/d)f'_c$ nor $(800\text{-}280a_v/d)$ psi.

(3) Reinforcement A_f to resist moment $[V_ua_v + N_{uc}(h\text{-}d)]$ shall be computed in accordance with Articles 2.31 and 2.32.

(4) Reinforcement A_n to resist tensile force N_{uc} shall be computed by $A_n = N_{uc}/\phi f_y$. Tensile force N_{uc} shall not be taken less than $0.2V_u$ unless special provisions are made to avoid tensile forces.

(5) Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$, or $(2A_{vf}/3 + A_n)$.

(d) Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5\ (A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

(e) Ratio $\rho = A_s/bd$ shall not be taken less than $0.04\ (f'_c/f_y)$.

(f) At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following:

(1) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;

(2) bending primary tension bars A_s back to form a horizontal loop, or

(3) some other means of positive anchorage.

(g) Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).

Article 2.29.7 (Page 8-2-34)

Add a new article 2.29.7 to read as follows:

2.29.7 Special provisions for brackets and corbels

(a) Provisions of Article 2.29.7 shall apply to brackets and corbels with a shear span-to-depth ratio a_v/d not greater than unity, and subject to a horizontal tensile force N_c not larger than V. Distance d shall be measured at face of support.

(b) Depth at outside edge of bearing area shall not be less than 0.5d.

(c) Section at face of support shall be designed to resist simultaneously a shear V, a moment $[Va_v + N_c(h\text{-}d)]$, and a horizontal tensile force N_c.

(1) Design of shear-friction reinforcement A_{vf} to resist shear V shall be in accordance with Article 2.29.4. For normal weight concrete, shear stress v shall not exceed $0.09f'_c$ nor 360 psi. For "all lightweight" or "sand-lightweight" concrete, shear stress v shall not exceed $(0.09\text{-}0.03a_v/d)f'_c$ nor $(360\text{-}126a_v/d)$ psi.

(2) Reinforcement A_f to resist moment $[Va_v + N_c\ (h\text{-}d)]$ shall be computed in accordance with Articles 2.26 and 2.27.

(3) Reinforcement A_n to resist tensile force N_c shall be computed by $A_n = N_c/f_s$. Tensile force N_c shall not be taken less than 0.2V unless special provisions are made to avoid tensile forces.

(4) Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$, or $(2A_{vf}/3 + A_n)$.

(d) Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5\ (A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

(e) Ratio $\rho = A_s/bd$ shall not be taken less than $0.04\ (f'_c/f_y)$.

(f) At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following:

(1) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;

(2) bending primary tension bars A_s back to form a horizontal loop, or

(3) some other means of positive anchorage.

(g) Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).

Article 2.35.5 (Page 8-2-43)

Revise article 2.35.5 to read as follows:

2.35.5 Horizontal shear design for composite concrete flexural members

(a) In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

(b) Design of cross sections subject to horizontal shear may be in accordance with provisions of Paragraph (c) or (d), or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests.

(c) Design horizontal shear stress v_{uh} at any cross section may be computed by

$$v_{uh} = \frac{V_u}{\phi\, b_v d} \tag{2-46}$$

where V_u is factored shear force at section considered and d is for entire composite section. Horizontal shear v_{uh} shall not exceed permissible horizontal shear v_h in accordance with the following:

(1) When contact surface is clean, free of laitance, but not intentionally roughened, shear stress v_h shall not exceed 80 psi.

(2) When minimum ties are provided in accordance with paragraph (e), and contact surface is clean and free of laitance, but not intentionally roughened, shear stress v_h shall not exceed 80 psi.

(3) When minimum ties are provided inaccordance with paragraph (e), and contact surface is clean, free of laitance, and intentionally roughened to a magnitude of approximately 1/4 in., shear stress v_h shall not exceed 350 psi.

(4) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by paragraph (e), permissible v_h may be increased by $160f_y/40{,}000$ psi.

(d) Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the actual change in compressive or tensile force, and provisions made to transfer that force as horizontal shear between interconnected elements. Horizontal shear shall not exceed the permissible horizontal shear stress v_h in accordance with paragraph (c) except that length of segment considered shall be substituted for d.

(e) Ties for horizontal shear

(1) A minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50b_v s/f_y$, and tie spacing s shall not exceed 4 times the least web width of support element, nor 24 in.

(2) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed). All ties shall be adequately anchored into interconnected elements by embedment or hooks.

AMERICAN RAILWAY ENGINEERING ASSOCIATION

Part 4
Pile Foundations

1990

4.1 GENERAL

4.1.1 Scope

(a) These specifications cover the investigation, design and construction of pile foundations.

(b) For the purpose of this part, a pile shall be considered as a relatively slender structural member continuously driven or augered into the earth. Drilled shafts placed in predrilled holes are addressed in Part 24, this chapter.

(c) In this specification, factors of safety are suggested; however, where information on loads or soil conditions is limited, larger factors of safety should be used.

4.2 DESIGN

4.2.1 Loads

(a) Pile foundations shall be designed to carry the entire superimposed load, including the weight of the footing and overlying loads supported by the footing.

(b) Pile foundations shall be designed for that reasonable combination of the following loads and forces which produce maximum load and in accordance with Article 4.3:

4.2.1.1 Primary Loads and Forces

(a) Dead

(b) Live - Vertical

(c) Live - Horizontal due to surcharge

(d) Centrifugal force

(e) Earth pressure

(f) Buoyancy

(g) Negative skin friction

NOTE: Live Load Impact shall be considered only in Case A of Article 4.2.2 for steel or concrete piles above the ground line where they are rigidly connected to the member supporting the superstructure.

4.2.1.2 Secondary Loads and Forces (Occasional)

(a) Wind and other lateral forces

(b) Ice and Stream flow

(c) Longitudinal forces

(d) Seismic forces

4.2.2 Loads on Piles

(a) Pile foundations shall be designed using the most restrictive of the following load capacity cases:

Case A: The capacity of an individual pile as a structural member

Case B: The capacity of the pile to transfer its load to the ground

Case C: The capacity of the ground to support the load from the pile or piles

When pile foundations are designed for primary and secondary loads in combination, as defined in Article 4.2.1, the allowable loads may be increased 25 percent for Load Cases A, B, and C, but the number of piles shall not be less than is required for primary forces alone with no increases in allowable stress for Case A and the minimum factor of safety shall be 2.0 for Cases B and C. For group friction piles, the factor of safety for Case C shall not fall below 2.0 for primary and secondary load combinations.

4.2.3 Eccentricity of Loads

(a) The maximum design pile load under eccentric loading shall not exceed the allowable load as determined under Article 4.3 with the appropriate factors of safety stipulated in Article 4.2.2. The piles shall be so spaced that the eccentric load on the piles, due to primary forces, will be distributed as equally as practicable to the piles in the group. Pile loads due to combinations of primary and secondary forces shall not exceed that permitted by Article 4.2.2.

4.2.4 Uplift on Piles

(a) In special cases when piles or pile groups are subjected to uplift, and sufficient bond and anchorage are provided between the pile and the supported structure, the uplift shall be considered in the design of the pile foundation. The pile foundation shall be designed for uplift considering load capacity Cases A, B, and C of Article 4.2.2. The factor of safety for Cases B and C shall be a minimum of 2.0 for combinations of primary and secondary forces, and a minimum of 3.0 for combinations of secondary forces with dead load alone. The capacity of the pile as a structural member (Case A) shall be based on allowable stresses established in the applicable AREA Specifications for Timber Structures, Chapter 7; in Part 2, this chapter; or in the AREA Specifications for Steel Structures, Chapter 15. The allowable stresses may be increased 25 percent for combinations of primary and secondary forces.

(b) The ultimate uplift value of an individual pile shall be determined by jacking test piles of identical type and dimension to that used in the design, and measuring the pull required per square foot of embedded surface area to raise the pile. When a tension pile group is involved, a group analysis shall also be undertaken. The maximum capacity of a tension pile group shall be considered to be the smaller of (1) the capacity of a single pile multiplied by the number of piles in the group, or (2) the weight of the block of soil contained within the perimeter of the groups, each with a minimum safety factor of 2.0, except as noted in paragraph (a).

4.2.5 Spacing of Piles

(a) Piles shall be spaced to nearly equalize their load consistent with economical design of the footings. The spacing of piles shall depend upon: the type of pile, that is whether friction or end bearing; the pile's structural and crushing strength; and the type of material sustaining the pile. Generally, piles should be spaced, center-to-center, at least three times the minimum butt width of the pile. Piles should be spaced far enough apart, or other suitable means used, to prevent heaving or uplifting of adjacent piles during driving.

(b) In small groups, the piles may be battered to enlarge the area sustaining the group, thereby increasing the load-carrying capacity of the group without unreasonably increasing the size of the foundation. End-bearing piles may be spaced in accordance with the capacity of the pile and the end-bearing stratum that will carry the design load. When closely spaced friction piles are contemplated, their total group capacity shall be verified by an acceptable geotechnical method which considers the capacity of the engaged soil mass to support the applied pile loads.

(3) When determining spacing of piles in granular soils, consideration should be given to the increased difficulty of driving due to the increased soil density that will occur because of soil packing or consolidation within the pile group.

4.2.6 Batter Piles

(a) Piles may be battered to help resist horizontal forces. Primary horizontal forces on pile foundations shall be resisted by batter piles where practicable. Such piles shall be designed to carry horizontal forces combined with their share of the vertical loads. In general, batter should not exceed 3 (horizontal) to 12 (vertical), due to increased difficulty in driving piles with a greater batter.

(b) Secondary horizontal forces on pile foundations may be accommodated by the shear resistance of the vertical piles, passive soil pressure, or friction between the soil/foundation interface where these resisting forces can be determined to exist for a particular foundation system. Where these resisting forces cannot be shown to be reliable over the expected life of the structure, batter piles or other dependable means of resisting these forces shall be used.

(c) Where large pile groups are involved, where clearance problems limit the pile foundation area, where secondary horizontal loads are small or in areas of the country where earthquake loading makes use of batter piles undesirable, the foundation shall be specially designed to include the horizontal forces as acting on the vertical piles. In such a case, the piles shall be designed to resist all loads, and the structure designed for the horizontal movement to be encountered.

4.2.7 Scour

(a) The possible effects of scour to pile foundations located in or adjacent to water should be reviewed as part of the total foundation design.

(b) When there is a possibility that the upper portion of the soil formations may be removed by scour, the piles or pile group shall be designed to have adequate bearing capacity and lateral support below the projected depth of scour. The free standing portions of the exposed piles shall be designed as columns.

(c) Determination of the probable depth of scour at a given location may have to be based largely on past records of stream bed erosion or wave action in the area, and how it has affected existing structures.

4.3 ALLOWABLE LOAD ON PILES

4.3.1 Subsurface Investigation

(a) Test borings shall be made at enough locations and to a sufficient depth below the anticipated tip elevation of the piles to determine adequately the character of the material through which the piles are to be driven and of the materials underlying the pile tips. The results of the borings and soil tests, taken into consideration with the function of the piles in service, will assist in determining the type, spacing, and length of piles that should be used and whether the piles will be end bearing, friction or a combination of both types.

(b) The subsurface investigation should be made in accordance with provisions outlined in Part 22, this chapter.

4.3.2 End Bearing Piles

(a) A pile may be considered end bearing when it passes through soil having low frictional resistance, and has its tip resting on relatively impenetrable material such as rock, or enters other material that offers rapidly increasing resistance to further penetration. The capacity of end-bearing piles depends on the bearing capacity of soil or rock material underlying the piles, and upon the structural capacity of the pile. The dynamic characteristics of the soil-hammer-hammer cushion-pile system coupled with the installation technique will determine the ability of the pile to penetrate overlaying strata to reach the bearing stratum.

(b) Allowable stresses for pile materials are given elsewhere in Part 4 of this chapter. When end-bearing piles pass through unconsolidated material, consideration should be given in design to the additional load (negative skin friction) that may be imposed on the pile as the material consolidates above the bearing stratum. The bearing stratum must be of sufficient thickness and strength to support the entire pile group loading. The design load shall preferably be determined by loading test piles. In addition, an analysis of the group of piles must show that the allowable load on the soil or rock supporting material is not exceeded.

4.3.3 Friction Piles

(a) A friction pile derives its support principally from the surrounding soil through the development of shearing or frictional resistance. The capacity of friction piles depends upon the ability of the soil to carry the load distributed by the piles within the limits of settlement that can be tolerated by the structure.

(b) The design load shall preferably be determined by loading test piles in accordance with the provisions of Article 4.3.6. Where groups groups of piles are driven into plastic materials, consideration should be given not only to the allowable load per pile, but also to the total load that can be safely assigned to the group. The design load shall be determined by loading a group of piles or by making an allowance for the difference between the capacity of a single pile and a group of piles by means of a block analsyis. A single row of piles need not be considered as a group, provided the piles are spaced at least three times their butt width.

(c) In many cases, a study of the borings and the estimation of approximate soil constants will determine the ability of the soil to carry the applied loads. In foundations involving cohesive soils, the load-settlement relationship should be investigated by recognized geotechnical methods and procedures.

4.3.4 Lateral Support

(a) A fully embedded pile can generally be considered laterally supported. A pile that is, or may be as a result of scour, in air or water, or which may be in muck, peat, thin mud, or fluid material, shall be investigated for the allowable capacity by the methods given in the Report of ACI Committee 543 "Recommendations for Design, Manufacture, and Installation of Concrete Piles" or other acceptable method approved by the engineer.

4.3.5 Pile Length Determination

(a) The determination of the most satisfactory and economical length of piles is one of the key factors in securing an adequate foundation. In addition to information that can be developed through soil borings, pile driving tests, pile load tests, and pile driving formulas, the use of the one-dimensional wave equation can be a valuable tool on large or difficult foundations, and is recommended for design and field control purposes. [1] Pile driving records of nearby adjacent piles may also be used in determining pile length if definite correlation between the existing and proposed piles as to type, loading, and use can be determined as well as the veracity of the previous pile driving record. The use of pile driving records to establish pile lengths without the benefit of a subsurface investigation and geotechnical analysis on projects which are not relatively small and where the conditions above cannot be met is not recommended.

4.3.5.1 Estimated Tip Elevation and Estimated Length

(a) At each boring location, using recognized geotechnical methods, the theoretical length of piles shall be computed considering contributions from both bearing and frictional resistance. Piles in very deep deposits are likely to receive support primarily through friction, whereas relatively shallow hardpan or rock conditions are likely to provide support primarily through end bearing. Many foundation conditions will provide both bearing and frictional support.

(b) At each individual boring, an estimated tip elevation and an estimated pile length shall be selected and tabulated based on the design cutoff elevation.

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[1] "Foundation Engineering," Peck, R. B., Hanson, W. E., and Thornburn, T. H., John Wiley & Sons, 2nd Ed., New York, 1974, Davisson, M. T., (1975), "Pile Load Capacity," Proceedings, Seminar Series, Design, Construction, and Performance of Deep Foundations, ASCE—U. of California, Berkeley.

4.3.5.2 Minimum Tip Elevation

(a) At each boring location, a minimum tip elevation shall be computed above which no structure piles will be permitted to stop. The minimum tip elevation reflects the design intent of the pile foundation design and is determined by an experienced foundation engineer's review of the estimated tip elevations, recognizing practical aspects of foundation construction practice. As an example, if geotechnical calculations demonstrate that piles should penetrate into a hardpan layer at varying depths, the minimum tip elevation will be shown at the top of this layer. In certain cases, field conditions during driving may modify this elevation.

(b) The minimum tip elevation will usually be above this estimated tip elevation.

4.3.5.3 Wave Equation

(a) The use of the one dimensional wave equation will greatly assist the engineer and contractor in determining the foundation adequacy and the construction of the project as planned.

(b) By the use of this tool, several values will be obtained: (1) the ability of the soil-hammer-hammer cushion-pile system to obtain the required design capacity, (2) the estimated blows per foot needed to obtain the required ultimate load at the estimated depth, (3) the means whereby the required blows per foot at other depths can be evaluated, (4) a means of evaluating the required blows per foot when the hammer fails to produce the manufacturer's rated energy.

(c) When this procedure is followed, the engineer can have the opportunity to modify his design before construction is started, and the contractor can be appraised of his hammer requirement.

(d) This procedure is recommended for all large and/or important projects.

4.3.5.4 Pile Driving Formulas

(a) Many dynamic pile driving formulas have been developed as an aid in determining pile capacities. While such formulas serve a useful purpose, particularly on smaller projects, greater accuracy, and economy can usually be obtained by use of the wave equation method as described in Article 4.3.5.3. If pile driving formulas are proposed for use, formulas that take into account the relationship between the weight of pile and weight of the pile hammer striking parts should be used.

4.3.5.5 Plan Tip Elevations

4.3.5.5.1 Friction Piles

(a) For those piles which can be considered to act as true friction piles, i.e., no end bearing stratum is in evidence within reasonable depths, only a design tip elevation is required. In uniform soils, where a complete soil investigation has determined the tip elevation, no further driving criteria is required, except the statement that the piles must be driven to the design tip elevation. A variation in the expected rate of penetration at the required tip elevation would indicate a variable soil layer, and a reappraisal of the tip elevation will be required.

4.3.5.5.2 Combined Bearing and Friction Piles, or Bearing Piles

(a) Plans and specifications should require that all piles be driven to the minimum tip elevation shown on the plans. At the minimum tip elevation, driving shall be continued until the required resistance is achieved, as determined by the load tests, a wave equation analysis, or some pile driving formula specified by the engineer. The latter provision will insure against variations in the consistency and depth of the bearing layer. An important judgement factor is selection of required hammer energy, type, and cushion. This decision can best be achieved by a wave equation analysis.

4.3.5.5.3 Estimated Lengths

(a) The plans should show estimated lengths which have been used for calculation of the engineer's estimate, and will provide the bidders with a reasonable basis for pricing the pile foundations.

4.3.6 Pile Driving and Loading Tests

4.3.6.1 Driving Tests

(a) Where variable soil conditions are known to exist, the following procedure is suggested. A few of the structure piles should be selected, including at least one from each substructure unit of the bridge or structure, and they should be driven first before other service piles are ordered. Their installations should be designated as Pile Driving Tests. A separate pay item should be provided, to cover piles installed by Pile Driving Tests.

(b) Where practicable, piles installed by driving tests should have their tips carried five to ten feet below the estimated tip elevation for the service piles at each particular location. Driving records for each foot of driving of each pile shall be kept and plotted in the field to provide exploratory information. The plot should be on a log containing the generalized information from the nearest structure boring. This record will provide an immediate correlation of driving resistance and subsoil conditions for the pile, hammer and cushion arrangement being used. The record will also provide information on where to select suitable locations for future load tests if required. (Load tests should be applied only to standard service piles, not to piles installed by pile driving tests. This is because piles installed by pile driving tests are deliberately overdriven and, therefore, are not typical of the service piles.)

(c) Piles installed by driving tests are recommended both for the situation where later load tests are to be performed and where load tests are not expected to be performed. The driving tests are of particular importance where load tests are not contemplated, because in that case, they provide the only correlation between soil boring data and driving data.

(d) If possible, piles installed by driving tests should be placed in a position where they can serve as service piles in the completed structure. It is permissible on small projects to over-drive all service piles (similar to the installation of driving tests described above) in lieu of load testing which, from a cost standpoint, may not be practical.

4.3.6.2 Pile Load Tests

(a) Pile load tests are considered essential for large or important jobs, or in subsurface conditions where there is little precedent for major construction. To date, they give the best knowledge of the probable capacity of an individual pile.

(b) It is preferred that load tests be carried to failure to determine the true factor of safety for the proposed design. If the margin of safety is higher or lower than desired, driving and elevation criteria can be modified. If, due to very high loads, tests to failure are not practicable, testing should be carried to not less than twice the design load. Test loads should not exceed the ultimate capacity of the pile as a structural member, or the capacity of the jack frame.

(c) The test apparatus and procedure shall be in accordance with the current ASTM Designation: D1143 "Standard Method of Testing Piles under Axial Compressive Load."

(d) By analyzing and interpreting the load tests with the driving test data and subsoil information, it will be possible to affirm the adequacy of the design and the installation criteria and introduce field modifications as may be necessary.

4.4 PILE TYPES

4.4.1 General

(a) Selection of the type of foundation pile for a particular structure should be based on the following criteria:

1. Design load per pile or pile group.
2. Type of foundation material to be penetrated.
3. Relative costs of the piles and pile driving.

4. Equipment available for driving piles.
5. Availability of desired pile type.
6. Special considerations based on specific job conditions, including, but not limited to:
    a. Restricted space for pile driving.
    b. Possible damage to existing structures.
    c. Exposure to sea water.
    d. Possible damage from marine organisms.
    e. Chemical attack.
    f. Possible damage to adjacent structures caused by vibration or soil movement during driving.
    g. Noise level during driving.

(b) Full-length piles shall be used wherever practicable, but if splices cannot be avoided, an approved method of splicing shall be used which will develop the full strength of the pile. Piles shall not be spliced except by permission of the engineer, who must also approve all splice locations.

4.4.2 Timber Piles

(a) Timber piles shall conform to the AREA specifications for wood piles, Chapter 7, Parts 1 and 3. If treatment is required, it shall conform to AREA specifications for wood preservation, Chapter 3, Part 6.

(b) For a timber pile which is primarily a friction pile, the maximum allowable load in pounds shall be computed by multiplying the tip area in square inches (small end) by the figure 1,200; the maximum load thus being equivalent to 1,200 psi acting at the tip.

(c) For a timber pile that is primarily a point bearing pile, the maximum allowable load shall be computed as above, but using the figure 800 instead of 1,200.

4.4.3 Steel Piles

4.4.3.1 Types

(a) This type of piling shall include all steel H-section piles and open-end steel pipe piles.

4.4.3.2 Material

(a) All steel used for the piles shall conform to the current ASTM Designations: A36 for H-pile sections, and A252 for pipe sections. Special steels may be used for corrosion protection or other purposes, but where welding is required, weldability must be assured.

4.4.3.3 Size

(a) The minimum depth of a steel H-section shall be 8 inches. The minimum thickness of metal in the flange or web shall be 3/8 inches. The flange width shall be not less than 85 percent of the depth of the section.

(b) The minimum outside diameter of open-end pipe piles shall be 8-5/8 inches. The minimum wall thickness shall be 3/8 inches.

4.4.3.4 Pile Caps

(a) In general, steel bearing caps are not required on steel H-piles embedded at least 1 ft. in concrete, providing the footing reinforcement is adequately designed to transmit the imposed loads.

4.4.3.5 Protection Against Corrosion

(a) Steel piles that will be exposed to corrosive environments shall be protected by concrete encasement or other suitable means; such as specially formulated epoxy or bituminous coatings, or additional steel thickness. Protection at ground surfaces or normal water lines shall be provided and shall extend at least 1 ft. above and 3 ft. below the ground surface or low-water line. Concrete protection, where provided shall have a minimum thickness of 4 inches and shall contain nominal steel reinforcement.

(b) Structural steel piles shall not be used through active corrosion-inducing material or where electrolysis may occur, without adequate provision for the protection of such piles.

4.4.3.6 Allowable Stresses

(a) The allowable load per pile shall be determined as specified in Section 4.3, but the unit stresses due to axial load shall not exceed 12,600 psi. Due allowance shall be made for any bending stresses caused by horizontal or eccentric loads and consideration shall be given to any column action of an unsupported pile.

4.4.3.7 Pile Tip Reinforcement

(a) Pile tip reinforcement may be required to prevent damage to H-piles when driving through heavy gravel, boulders, or formations known to contain obstructions, or when driving end bearing piles. Heavy cast steel tips are recommended for this purpose where the conditions so justify their use.

4.4.4 Precast Concrete Piles

4.4.4.1 General

(a) This type of piling includes both conventionally reinforced concrete piles and prestressed concrete piles. Both types can be formed by either casting, centrifugal casting, or extrusion methods. They are made in various cross section shapes such as square, octagonal, and round. Often such piles are cast with a hollow core. The piles are usually of constant cross section but may have a tapered tip.

(b) Precast concrete piles must be designed and manufactured to withstand handling and driving stresses in addition to service loads. The workmanship, material, and proportioning shall conform to the requirements specified in Part 1, this chapter.

4.4.4.2 Design

(a) The minimum acceptable diameter or side dimension for driven piles is usually 8 inches. This may be satisfactory for short piles which are lightly loaded, however, as a general rule, it is recommended that the minimum average dimension be 10 inches, except that the pile tip may be 8 inches. Piles may be pointed or not as directed by the engineer.

4.4.4.3 Manufacture

(a) The manufacture of the various types of precast concrete piles shall be in accordance with the current Chapter 4 of American Concrete Institute (ACI) Committee 543 report titled “Recommendations for Design, Manufacture, and Installation of Concrete Piles.”

4.4.4.4 Cut-Off

(a) Precast piles shall be driven to or cut off within 2 inches of the elevation shown on the plans, but in all cases, the cut-off shall be below any indication of fracture. If piles are cut off or driven below the required elevation, they shall be built-up to the cut-off line as determined by the engineer. Standard details are to be shown on the project plans.

4.4.4.5 Allowable Stresses

(a) The allowable load per pile shall be determined as specified in Section 4.3, but the concrete unit stresses shall not exceed $0.3\ f'_c$ with a maximum of 1,600 psi. Other stresses shall conform to the requirements of Parts 2 and 17, this chapter.

4.4.5 Cast-In-Place Concrete Piles

4.4.5.1 Types

(a) Cast-in-place piles shall be cast in previously driven metal casings or shells which shall remain permanently in place. They may be tapered or cylindrical, or a combination of tapered and cylindrical shapes.

4.4.5.2 Tapered Piles

(a) Tapered piles shall not be less than 8 inches in diameter at the tip and shall be uniformly tapered at the rate of not more than 1 inch in 8 ft., or step tapered, at the same average rate.

4.4.5.3 Cylindrical Piles

(a) Cylindrical piles shall have a minimum diameter of 8 inches.

4.4.5.4 Pipe Casings and Shells

(a) Pipe casings driven without a mandrel shall be formed of steel conforming to the current ASTM Designation: A252, Grade 2. Metal shells driven with a mandrel shall have a thickness of not less than No. 16 USMSG and a minimum yield strength of 30,000 psi. Casings shall be in one integral piece or adequately spliced, and shall be of sufficient thickness to withstand installation pressures without leakage or harmful distortion.

(b) All piles shall be equipped with approved watertight flat plates or conical points welded to the tip end of the casing. The end closures approved for cylindrical piles shall not project beyond the diameter of the pile casing when used on friction piles.

4.4.5.5 Placing Concrete

(a) Casings or shells shall be inspected and approved by the engineer immediately before any concrete is placed. A suitable light shall be used to inspect the entire length. Any accumulated foreign matter, or water shall be removed before the concrete is placed. Any broken or otherwise defective shells shall be corrected by removal and replacement, or by driving an additional pile, as directed by the engineer. Concrete having a minimum compressive strength of at least 2,500 psi at 28 days shall be used to fill the shell. The placing of the concrete shall be carried out as a continuous operation from the tip to the cut-off elevation, and shall be performed in such a manner as to minimize segregation and insure complete filling of the casing or shell.

(b) No pile shall be driven within 15 ft. of a pile that has been filled with concrete for more than 2 hours but less than 24 hours. The driving procedure for any particular project shall be approved by the engineer in charge, before commencing work.

4.4.5.6 Allowable Stresses

(a) The allowable load per pile shall be determined as specified in Section 4.3., but the unit stresses, shall not exceed the following:

Concrete: 0.3 of the ultimate compressive unit strength of the concrete used (f'_c), but not to exceed 1,600 psi.

Steel: The unit stresses shall not exceed 12,600 psi.

4.4.5.7 Protection Against Corrosion

(a) When the steel casing is used in computing the strength of the pile and the piles will be exposed, they shall be protected from corrosion as specified in Article 4.4.3.6.

(b) If the strength of the steel is considered in computing the strength of the pile, the pile shall not be used through active rust-inducing material or where electrolysis may occur without adequate provision for the protection of such pile.

4.4.5.8 Reinforcement

(a) Cast-in-place piles may be reinforced to provide needed bending strength, or for uplift anchorage. When used, the reinforcing steel should be preassembled into cages and accurately placed in accordance with design drawings. The reinforcement shall be clean of foreign material that could affect bond, and securely positioned before concrete fill is placed.

4.4.6 Augered Cast-In-Place Piles

4.4.6.1 General

(a) Augered Cast-In-Place piles are primarily used as friction piles. They are installed by rotating a continuous flight hollow-shaft auger into the ground to a predetermined pile depth. High-strength mortar is pumped with sufficient pressure as the auger is withdrawn, to fill the hole preventing hole collapse and causing the lateral penetration of the mortar into soft or porous zones of the surrounding soil. A head of at least several feet of mortar above the injection point is carried around the perimeter of the auger at all times during the raising of the auger so that the high strength mortar has a displacing action removing any loose material from the hole.

4.4.6.2 Design

(a) The length of pile will be determined from the examination of soil borings using the shear strength of the soil, and preferably, verified by pile load tests as described in Article 4.3.6.2. Recommended pile loads for varying pile diameters, depending on soil strengths, are given in the chart below:

Nominal Size of Pile (Inches)	Normal Loadings Range (Tons)	Normal Required Compression Strength of Mortar (PSI)
12	10-40	2,000-2,500
14	40-75	2,500-3,000
16	75-100	3,000-4,000

4.4.6.3 Materials

(a) The material used to fill the holes shall consist of a mixture of Portland Cement, concrete sand, fluidizer and water proportioned and mixed as to provide a mortar capable of maintaining the solids in suspension without appreciable water gain and which will laterally penetrate and fill any voids in the foundation material. Portland Cement shall conform to this Chapter, Article 1.2, Cement Type I. The fine aggregate shall conform to this Chapter, Article 1.3, with a fineness modulus between 1.40 and 3.40. Fluidizer shall meet the requirements of the current Corps of Engineers, USA, Spec. No. CRDC 566.

(b) The mortar shall be so proportioned as to have a minimum ultimate compressive strength of 2,000 psi at 28 days. A set of 6 mortar cubes shall be made each day and tested in accordance with the current ASTM Designation: C109, with the exception that the mortar should be restrained from expansion by a top plate.

4.4.6.4 Tension Piles

(a) Where tension is required, a special continuous flight hollow-shaft auger shall be rotated into the ground to the required depth. A steel bar shall be inserted into the hollow center shaft of the auger. The auger head closure shall be detached allowing the steel bar to remain in place and be centered in the tension pile as the continuous flight auger is slowly withdrawn from the hole. During this withdrawal process, high strength mortar shall be placed under pressure through the space between the steel rod left in place and the wall of the hollow shaft auger.

4.5 INSTALLATION OF PILES

4.5.1 Driven Piles

(a) Piles shall be driven with steam, air, or diesel powered hammers. Size of the type of hammer used should be determined by guidelines noted in Article 4.5.1.1. The hammer shall be operated at all times at pressures and speeds recommended by the manufacturer. Use of vibratory type hammers may be allowed if pile capacities have been determined by load tests. Use of a gravity drop hammer for driving piles should be limited to relatively unimportant foundations where uniform pile capacity is not critical.

4.5.1.1 Selection of Hammer-Cushion Combination

(a) Preliminary selection of the hammer-cushion combination for driving piles can be made with the following guide:

Steel Piles—Air or Steam Operated Hammers

Minimum size: 170 ft.-lb. of rated energy per ton of pile service load. Stiff or hard internal cushion.

Desired size: 250-340 ft.-lb. of rated energy per ton of pile service load.

Pile Cushion: Moderately stiff to soft (wood) internal cushion.

Diesel Hammers:
Use 100 to 135% of size determined for air or steam hammers.
Use standard (stiff) internal cushion.

Mandrel-Driven Piles—Same as Steel

Precast or Prestressed Concrete Piles—Air or Steam Operated Hammers

Desired size: 250 ft.-lb. of rated energy per ton of pile service load. The weight of the ram shall generally not be less than one-fourth of the weight of the pile being driven. Use wood, or equivalent, internal cushion.

Pile Cushion: Design by one-dimensional wave theory, or by experience.

Diesel Hammers:

Use 100 to 135% of size determined for air or steam hammers.
Use standard (stiff) internal cushion.

Wood Piles—Air or Steam Operated Hammers

For normal capacity piles (up to 30 tons service load) excluding abnormally large timbers or fabricated sections, use 15,000 ft.-lb. maximum rated energy with a wood internal cushion. Diesel hammers may be rated up to 20,000 ft.-lb. with standard (stiff) internal cushions.

(b) The foregoing preliminary selection of hammer and cushion combinations should preferably be confirmed by a wave equation analysis of pile driving indicating the pile yield stresses are not exceeded and that the desired ultimate load capacity can be achieved (see Section 4.3.5.3).

4.5.1.2 Pile Leads

(a) Pile drivers shall have firmly supported leads extending from the highest point to the lowest point that the hammer must travel. The leads should be supported independently of the pile and constructed to guide and stay the pile during driving.

4.5.1.3 Splicing

(a) Driving shall be continued until the plan tip elevation is reached or until the rate of penetration specified is obtained. If the proper resistance to driving is not attained at the plan cut-off, the driving shall be continued and the additional length of pile required shall be supplied by splicing in such a way as to develop the full strength of the section of the pile. The splice shall be made a sufficient distance, but not less than 1 ft. above the ground or water surface so that the splice can be observed during subsequent driving.

4.5.1.4 Jetting

(a) Piles may be jetted, when permitted by the engineer, either by use of water jets alone or in combination with the hammer. The volume and pressure of the water at the jet nozzles shall be sufficient to freely erode the material adjacent to the pile. Before the desired penetration is reached, jetting shall be discontinued at the elevation specified by the engineer and the piles driven to required penetration or resistance.

4.5.1.5 Preboring

(a) Where piles must be installed through strata offering high resistance to driving, where jetting would cause damage, to prevent excessive heaving of cohesive soils, for driving through relatively impenetrable material or for other valid reasons, the engineer may require or permit holes to be bored with a power auger or other equipment especially designed for the purpose. Depending upon the reasons for preboring, the diameter of the hole shall be as directed by the engineer to obtain the proper pile penetatration and carrying capacity. The pile shall be inserted into the hole immediately after boring and be driven to required penetration or resistance.

4.5.1.6 Improperly Driven and Damaged Piles

(a) Piles shall be driven within 3 inches of the plan location. Variations of more than 1/4 inch per ft. from the vertical, or from the batter line when batter piles are required, may be subject to rejection by the engineer. Any pile so out of line or plumb as to impair its usefulness shall be pulled and/or an additional pile driven, as required by the engineer. Any pile so injured in driving or handling as to impair its structural capacity as a pile under conditions of use shall be replaced by a new pile, or the injured part shall be replaced by splicing or other remedial measures—all as directed by the engineer.

4.5.1.7 Redriving of Heaved Piles

(a) Previously driven piles shall be carefully checked during the driving of adjacent piles, and if any uplift occurs, they shall be redriven to the required penetration or resistance as directed by the engineer.

4.5.1.8 Underwater Driving

(a) While it is impossible to drive piles underwater by use of a follower between the pile and hammer, or by use of a submersible pile hammer, such driving methods should be avoided when it is necessary to drive piles to obtain a predetermined bearing capacity, unless such capacity is determined by a pile load test under similar conditions.

4.5.1.9 Interrupted Driving

(a) When driving is interrupted or the rate of blows retarded for any reason, a careful record shall be kept of the extent of the delay or retardation. Any decrease in the penetration per blow immediately following such stoppage, shall be cause to suspect the interpretation of the preceding blows per foot.

4.5.2 Augered Cast-In-Place Piles

4.5.2.1 Augering Equipment

(a) The hole through which the high-strength mortar is pumped during the placement of the pile shall be located at the bottom of the auger head below the bar containing the cutting teeth.

(b) The auger flighting shall be continuous from the auger head to the top of auger with no gaps or other breaks. The pitch of the auger flighting shall not exceed 9 inches.

(c) Augers over 40 ft. in length shall contain a middle guide. The piling leads should be prevented from rotating by a stabilizing arm.

4.5.2.2 Mixing and Pumping of High-Strength Cement Mortar

(a) Only approved pumping, continuous mixing, and agitating equipment shall be used in the preparation and handling of the mortar. All oil or other rust inhibitors shall be removed from mixing drums and mortar pumps. If ready-mix mortar is used, an agitating storage tank of sufficient size shall be used between the ready-mix truck and the mortar pump to insure a homogeneous mix and continuity in the pumping operations. All materials shall be such as to produce a homogeneous mortar of the desired consistency. If there is a lapse in the operation, the mortar shall be recirculated through the pump.

(b) The mortar pump shall be a positive displacement piston type pump capable of developing displacing pressures at the pump of up to 350 psi.

4.5.2.3 Pile Top Encasement

(a) Metal sleeves or casing of the proper diameter and at least 18 inches in length shall be placed around the pile tops. (Special conditions may require metal sleeves of additional length.)

4.6 INSPECTION OF PILE DRIVING

(a) Pile driving and augering operations shall be inspected and documented completely as directed by the engineer. Recommended techniques of inspection and records to be compiled can be found in the publication titled "Inspection of Pile Driving Operations" Technical Report M-22, Department of the Army, Construction Engineering Research Laboratory, Champaign, Illinois, July 1972.

Proposed 1990 Manual Revisions To Chapter 10—Concrete Ties

The following changes are proposed to two articles in Chapter 10:

It is proposed that Article 1.2.3.1 be replaced by the following:

1.2.3.1

Strands for pretensioning tendons shall conform to ASTM A 416, "Specification for Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete," or ASTM A 886, "Specification for Indented Seven-Wire Stress-Relieved Strand for Prestressed Concrete." Wires for pretensioning tendons shall conform to ASTM A 421, "Specification for Uncoated Stress-Relieved Wire for Prestressed Concrete," or ASTM A 864, "Specification for Steel Wire, Deformed, for Prestressed Concrete Railroad Ties."

Strands other than those listed in ASTM A 416 or A 886 and wires other than those listed in ASTM A 421 or A 864 may be used provided they conform to the requirements of these specifications and have no properties which make them less satisfactory than those listed in these specifications.

It is proposed that Article 1.2.3.2 be replaced by the following:

1.2.3.2

Steel bars for posttensioning tendons shall conform to ASTM A 722, "Specification for Uncoated High-Strength Steel for Prestressed Concrete." Bars of other designations may be used provided they conform to the requirements for yield strength, tensile strength, and elongation, stipulated in ASTM A 722.

Proposed 1990 Manual Revisions To Chapter 11—Engineering Records and Property Accounting

Changes proposed to Chapter 11 specifications are as follows:

Foreword (Page ii)

The second to last paragraph of the foreword is to be replaced with:

In keeping with the International Membership of the Committee, Part 1 of the Chapter includes a comparison of the differences which currently exist between the system of property accounts as prescribed by the National Transportation Agency of Canada and that of the Interstate Commerce Commission.

Table of Contents (Page ii)

Under Part 1: Accounting, add Article "1.11 National Transportation Agency of Canada System of Accounts"

Index—Part 1 (Page 11-1-1)

To Article 1.4 add:

—Account 52 Locomotives
—Account 53 Freight-Train Cars
—Account 54 Passenger-Train Cars
—Account 55 Highway Revenue Equipment

—Account 56 Floating Equipment
—Account 57 Work Equipment
—Account 58 Miscellaneous Equipment

After Article 1.10 add:

1.11 National Transportation Agency of Canada System of Accounts

Article. 1.1.2 (Page 11-1-2)

Replace current article with the following:

1.1.2 The accounts prescribed by the National Transportation Agency of Canada are discussed in Section 1.5.

Article 1.1.3 (Page 11-1-2)

Replace subarticle (a) with the following:

(a) To explain road property and rolling stock accounts and to define construction and retirement activities applicable to each account.

Article 1.4.30 Account 59 (Page 11-1-11)

Renumber this article to "1.4.37 Account 59," and insert new Articles 1.4.30 to 1.4.36 as follows:

1.4.30 ACCOUNT 52 LOCOMOTIVES

Definition

The locomotives account includes the cost of locomotives and tenders including the appurtenances and fixtures necessary to equip them for service as well as the cost of transportation to the carrier's line and the inspection, setting up and testing after receipt from builders. The cost of spare engines carried on hand for the purpose of temporarily replacing similar units during repairs or overhauls is also included.

Typical Items Included	To Be Reported
Locomotives—all types	Reporting marks
Tenders	Unit number
Spare engines	Unit type
Pneumatic Sanding equipment	Horsepower
Radio equipment (permanently attached)	Builder
	Year built
	Installed status (new, used or rebuilt)
	ICC valuation group
	Financial obligation

1.4.31 ACCOUNT 53 FREIGHT-TRAIN CARS

Definition

The freight-train cars account includes the cost of freight-train cars of all types including the appurtenances and fixtures necessary to equip them for service as well as the cost of inspection and transportation to the carrier's line.

Typical Items Included	To Be Reported	
Freight-train cars—all types	Reporting marks	Installed status (new, used or rebuilt)
Loading devices	Unit number	ICC valuation group
Refrigeration units	Unit type	AAR car type code
	Builder	Financial obligation
	Year built	

1.4.32 ACCOUNT 54 PASSENGER-TRAIN CARS

Definition

The passenger-train cars account includes the cost of passenger-train cars of all types including the appurtenances, furniture, and fixtures necessary to equip them for service as well as the cost of inspection and transportation to the carrier's line.

Typical Items Included

- Passenger-train cars—all types
- Automobile carriers
- Baggage cars
- Fixtures and floor coverings
- Seats

To Be Reported

- Reporting marks
- Unit number
- Unit type
- Horsepower
- Builder
- Year built
- Installed status (new, used or rebuilt)
- ICC valuation group
- Financial obligation

1.4.33 ACCOUNT 55 HIGHWAY REVENUE EQUIPMENT

Definition

The highway revenue equipment account includes the cost of highway vehicles used in all revenue transportation service including the appurtenances necessary to equip them for service as well as the cost of the inspection and transportation required for delivery of the vehicles into the carrier's revenue service.

Typical Items Included

- Buses
- Containers
- Semi-trailers
- Tractors
- Trailers
- Trucks
- Radio communication equipment

To Be Reported

- Reporting marks
- Unit number
- Unit type
- Model number
- Serial number
- Builder
- Year built
- Installed status (new, used or rebuilt)
- ICC valuation group
- Financial obligation

1.4.34 ACCOUNT 56 FLOATING EQUIPMENT

Definition

The floating equipment account includes the cost of marine or floating equipment of all kinds including the appurtenances and fixtures necessary to equip them for service as well as the cost of inspection and transportation to the carrier's line. Work equipment is not included.

Typical Items Included

- Barges
- Ferryboats
- Power launches
- Transfer boats
- Tugboats

To Be Reported

- Reporting marks
- Unit number
- Unit type
- Builder
- Year built
- Installed status (new, used or rebuilt)
- ICC valuation group
- Financial obligation

1.4.35 ACCOUNT 57 WORK EQUIPMENT

Definition

The work equipment account includes the cost of work equipment of all kinds including the appurtenances, furniture and fixtures necessary to equip them for service as well as the cost of inspection and transportation to the carrier's line.

Typical Items Included

- Ballast cars
- Business cars
- Camp cars
- Derrick cars
- Dynamometer cars
- Training cars
- Rail cars
- Tool cars

To Be Reported

- Reporting marks
- Unit number
- Unit type
- Builder
- Year built
- Installed status (new, used or rebuilt)
- ICC valuation group
- AAR car type code
- Financial obligation

1.4.36 ACCOUNT 58 MISCELLANEOUS EQUIPMENT

Definition

The miscellaneous equipment account includes the cost of automobiles, trucks and other highway equipment not used in revenue transportation service and not provide for elsewhere including the appurtenances necessary to equip them for service as well as the cost of the inspection and transportation required for delivery of the vehicles to the carrier. The cost of on and/or off-track automotive vehicles permanently equipped with special purpose machinery and used exclusively in maintenance of way and structures is not included.

Typical Items Included

- Airplanes
- Automobiles
- Buses
- Emergency repair vehicles
- Tractors
- Trailers
- Radio communication equipment

To Be Reported

- Reporting marks
- Unit number
- Unit type
- Model number
- Serial number
- Builder
- Year built
- Installed status (new, used or rebuilt)
- ICC valuation group
- Financial obligation

Article 1.11 (Page 11-1-13)

Add the following new material:

1.11 NATIONAL TRANSPORTATION AGENCY OF CANADA SYSTEM OF ACCOUNTS

1.11.1 There are many similarities between the ICC primary accounts and the system accounts prescribed by the National Transportation Agency of Canada. However, there also exist some major differences. The following table compares NTA accounts with the ICC accounts and illustrates the differences.

TABLE 1.11.1

No.	NTA Title	No.	ICC Title	Remarks Pertaining to NTA Accounts
101	Land	2	Land for transportation purposes	
102	Grading	3	Grading	
103	Rail	9	Rail and Other Track Material	Includes only that portion of cost applicable to rail material and welding
105	Ties	8	Ties	Includes only that portion of cost applicable to tie material
107	Other Track Material	9	Rail and Other Track Material	Includes only that portion of cost applicable to track material
109	Ballast	11	Ballast	Also includes the cost of labor used in laying ballast
111	Track Laying and Surfacing	N/A		This account includes the cost of labor incurred installing the material items included in Accounts 103, 105 and 107 in new track.
113	Used Track Material in Store	N/A		Material in this account is included in ICC balance sheet account 712, Material and Supplies.
115	Bridges	6	Bridges, Trestles and Culverts	Includes only that portion of cost applicable to the construction of bridges and trestles
117	Culverts	6	Bridges, Trestles and Culverts	Includes only that portion of cost applicable to the construction of culverts
119	Tunnels	5	Tunnels and Subways	
121	Fences, Snow Sheds and Rock Sheds	13	Fences, Showsheds and Signs	
123	Public Improvements	7 39	Elevated Structures Public Improvements—Construction	
125	Other Right-of- Way Property	4	Other Right-of-Way Expenditures	
127	Other Expenditures—Road	N/A		This is an historic account which is not to be used for new investment.
131	Office and Common Buildings	16 29	Stations and Office Buildings Power Plants	Does not include the cost of buildings used as passenger stations or freight terminals nor the cost of moveable equipment and machinery

133	Offices and Common Buildings Moveable Machinery	16	Stations and Office Buildings	Stations and Office Buildings includes only the cost of moveable equipment and machinery
		31	Power Transmission Systems	Includes only the cost associated with multi-purpose power plants
		45	Power Plant Machinery	
		59	Computer Systems and Word Processing Equipment	
135	Passenger Stations	16	Stations and Office Buildings	Including only the cost of passenger stations not including moveable machinery and equipment
137	Passenger Station Moveable Equipment	16	Stations and Office Buildings	Includes only the cost of moveable equipment installed in passenger stations
139	Roadway Buildings	17	Roadway Buildings	Does not include the cost of moveable machines and equipment installed in buildings.
141	Roadway Building Machines and Moveable Machinery	17	Roadway Buildings	Includes only the cost of moveable machines and equipment installed in buildings
143	Equipment Repair Shops	20	Shops and Enginehouses	
145	Shop Machinery and Moveable Equipment	44	Shop Machinery	
147	Leasehold Improvements	N/A		Items in this account are included in ICC balance sheet account 732, Improvements to Leased Property, and are further classified by the primary accounts used for account 731.
149	Signals	27	Signals and Interlockers	
151	Rail Communication Systems	26	Communication Systems	
153	Electrified Rail Systems	31	Power Transmission Systems	Includes only the cost for systems for supplying power for electrified rail systems; the cost of power transmission systems associated with a multi-purpose power plant is included in Account 133.
157	Intermodal	25	TOFC/COFC	Does not include the cost of freight handling equipment
159	Rail Freight Terminals	16	Stations and Office Buildings	Includes only the cost of freight terminals not including freight handling equipment

161	Marine Terminals	23 24	Wharves and Docks Coal and Ore Wharves	
163	Fuel Stations	19	Fuel Stations	
171	Locomotives	52	Locomotives	
173	Freight Cars	53	Freight Train Cars	
175	Passenger Cars	54	Passenger Train Cars	
177	Intermodal Terminal Handling Equipment	25	TOFC/COFC Terminals	Includes only the cost of freight handling equipment
179	Trailers, Semi-trailers, Chassis and Bogies	55	Highway Revenue Equipment	Includes only the cost of trailers, semi-trailers, chassis and bogies used in intermodal service
181	Highway Tractors	55	Highway Revenue Equipment	Includes only the cost of highway tractors used in intermodal service.
183	Roadway Machines	37	Roadway Machines	Does not include the cost of machines valued over one million dollars
185	Large Scale Roadway Equipment	37	Roadway Machines	Includes the cost of roadway machines valued over one million dollars
187	Work Equipment	57	Work Equipment	Includes the cost of equipment used for the purpose of maintaining, improving and/or constructing way and structures
189	Other Non- Revenue Rolling Stock	57	Work Equipment	Includes the cost of equipment used for purposes other than revenue service or maintaining, improving and/or constructing way and structures
191	Marine Equipment	56	Floating Equipment	
193	Buses	58	Miscellaneous Equipment	Includes only the cost of used in revenue transportation service
195	Miscellaneous Equipment	58	Miscellaneous Equipment	Includes the cost of trucks, automobiles and other vehicles not included elsewhere

Note 1: Items included in ICC Account 35, Miscellaneous Structures, are to be included in the appropriate NTA account.

Note 2: ICC Account 22, Storage Warehouses, has no NTA counterpart.

Note 3: Items included in ICC Account 18, Water Stations, are included as part of the facility at which they are located.

Proposed 1990 Manual Revisions To Chapter 12—Rail Transit

A Table of Contents and "Check List for Passenger Facilities" is proposed in the initial development of Chapter 12 material. The proposed text is as follows:

TABLE OF CONTENTS

Part 1—Rail Transit—General Information

- **1.1** Description of Rail Transit Systems
- **1.2** Role of the Sponsoring Agencies in Transit Activities
- **1.3** Interfaces With Other AREA Chapters

Part 2—Rail Transit—Corridor Planning Considerations

- **2.1** Introduction
- **2.2** Corridor Goals
- **2.3** Data Collection
- **2.4** Corridor Identification
- **2.5** Corridor Evaluation
- **2.6** Rail Transit Technology Selection
- **2.7** Regional Transportation Interfaces
- **2.8** Corridor Implementation Strategies

Part 3—Track and Roadway Considerations

- **3.1** Introduction
- **3.2** Design Considerations
- **3.3** Maintenance Considerations
- **3.4** Special Considerations
- **3.5** Portfolio/Plans

Part 4—Bridge and Structural Considerations

- **4.1** Introduction
- **4.2** Stations
- **4.3** Multi Mode Facilities
- **4.4** Shop and Service Facilities
- **4.5** Bridge and Drainage Structures
- **4.6** Tunnels and Subways
- **4.7** Concrete Slab Track

Part 5—Electrification Systems

- **5.1** Introduction
- **5.2** Power Requirements—Equipment & System

5.3 Power Source and Supply

5.4 Power Equipment

5.5 Special Considerations

The following proposed check list will eventually be incorporated into Part 4—Bridge and Structural Considerations.

1. Station Location
    A. Community Considerations
    B. Ease of Access, Highway, Pedestrian, etc.
    C. Provision for Expansion
    D. Operational Visibility
    E. Track Geometry Consideration
    F. Joint Development
2. Intermodal
    A. Transfers
    B. Private Vehicle Parking
    C. Traffic Circulation
3. High vs. Low Platforms
    A. Traffic Volume
    B. Clearance Considerations
    C. Sizing
4. Passenger Flow
    A. Walkways
    B. Stairs
    C. Elevators
    D. Escalators
    E. Moving Walkways
5. Elderly and Handicapped Considerations
    A. Access
    B. Location
    C. Specialized Facilities
6. Fare Collection
    A. Free-Paid Areas
    B. Attendant Facilities
    C. Fare Vending Machines
    D. System Related Items
7. Climate Considerations
    A. Heating/Cooling
    B. Shelters

8. Lighting
    A. Interior
    B. Platform
    C. Pick-up and/or Parking Areas
9. Utilities
    A. Consumption
    B. Facilities
    C. Panels and Cabinets
10. Passenger Facilities
    A. Rest Rooms
    B. Seating
    C. Public Phones
    D. Vending Machines/Concessions
    E. Trash Containers
11. Security
    A. Fences, Gates, Barriers, Railings
    B. Closed Circuit T.V. Surveillance
    C. Police Facilities
12. Fire Protection
    A. Fire Codes
    B. Types of Equipment
13. Communications
    A. Public Address
    B. Automated/Controlled Information Signs
    C. Passenger Aid Systems
14. Vandalism and Graffiti Protection
    A. Security
    B. Building Layout
    C. Materials
15. Signs
    A. Type, Standards
    B. Informational Signs
    C. Directional Signs
    D. Instructional Signs
16. Advertising
    A. Policy
    B. Location
    C. Size and Type
    D. Joint Usage

17. Landscaping
    A. Amount and Type
    B. Transit or Community Responsibility
18. Emergency Considerations
    A. Access for Emergency Vehicles
    B. Access for Emergency Personnel
    C. Special Considerations for Subway and Elevated Stations
    D. Instruction for Emergency Personnel
    E. Evaluation
19. Safety Evaluation
    Note—Details for this section to be done at a future date.

Proposed 1990 Manual Revisions To Chapter 13—Environmental Engineering

Current Section 5.6 on "Diesel Locomotive Fuel and Water Services" will be replaced by a new Section 5.6 on "Fuel and Lubricating Oil Systems." The new Section 5.6 takes into account the latest technology for handling diesel fuel. It drops previous Articles 5.6.8—diesel watering facilities, 5.6.9—diesel cooling water, 5.6.10—steam generator water and 5.6.11 service area paving because of their minor environmental significance. In the future other sections of Part 5—Plant Utilities when revised will emphasize more of environmental consequences associated with plant utilities.

5.6 FUEL AND LUBRICATING OIL SYSTEMS

5.6.1 General Principles

5.6.1.1 General

(a) The assignment of Subcommittee 5 of Committee 13 is to prepare manual material discussing fuel and lubricating oil systems in light of environmental considerations. The products discussed are used by most railroads in large quantities. These products, due their inherent nature and the nature of constituents assimilated during use, can be environmentally damaging. It is not the intent of this committee to tell the railroads which products to use, but rather to discuss procedures used for handling these products in an environmentally responsible manner.

(b) Laws and/or Regulations

There are several governing bodies regulating the storage and use of petroleum products. These agencies and their basic regulatory mission are as follows:

Note: The following paragraphs identify several CFR and other references; the most current publication should be obtained and followed.

1. State or Local Fire Marshall

The State and/or Local Fire Marshall should be contacted when installing any petroleum handling facility. They will be concerned that the facilities meet the requirements from a fire protection standpoint. They will consider diking and proper electrical requirements. The National Fire Protection Association has a publication entitled NFPA-30 "Flammable and Combustible Liquids" code addressing these requirements.

2. US, State, and Local EPA

(a) The USEPA, State, and Local Environmental Agencies have requirements for design of facilities. EPA regulation, 40 CFR 112, describes recommended procedures, methods, and equipment to prevent discharges of oil from handling facilities. These regulations describe the requirements for Spill Control and Countermeasure (SPCC) plans. A SPCC plant must be prepared for each facility and a copy must be maintained at the site for all petroleum handling facilities which meet specific size requirements. The recommendations include diking and spill containment measures. In some states, these requirements are administered by local agencies such as counties and water boards.

(b) The USEPA and most state and local agencies also have requirements for underground storage tanks. These include the tank compatibility and integrity requirements along with monitoring requirements for the tank. The federal regulations can be found in 40 CFR 280. These regulations include secondary containment for piping, partially buried tanks, etc. In addition, state or local regulatory authorities may have requirements that are somewhat different or more strict. Registration of underground storage tanks is mandatory under USEPA and state regulations. These regulations are generally administered by the individual states.

(c) In the handling of petroleum products, spills can and do occur. Generally speaking, these spills are collected in either dedicated systems for that product, or in sewers which service railroad facilities. USEPA, state, and local authorities have requirements for discharge from railroad facilities. For more detailed discussion, refer to Chapter 13, Part 1.2—Industrial Wastewater Treatment and Disposal.

(d) Generally, there are no USEPA regulations controlling air pollution of #2 diesel fuel, however, state and local regulations may apply. Products with a high vapor pressure, such as gasoline, do have regulation requirements, however.

3. Coast Guard

Where fuel is to be unloaded from barges or other water borne transportation, approval from the Army Corps of Engineers and/or Coast Guard should be obtained. Coast Guard regulations on oil spills can be found in 33 CFR 153-157.

5.6.1.2 Characteristics

(a) General

Petroleum products include diesel fuel oil, crankcase lubricating oil, hydraulic fluids, journal oil, kerosene (#1 Diesel), grease, rail lubricants, and gasoline. Design of the facilities should take into account the characteristics associated with these products. It should be noted that characteristics can vary depending upon your specific supplier, the geographic location (or source of product), and the specifications for the product. Manufacturers should be contacted for product information and material safety data sheets for their respective products.

5.6.2 Additives

5.6.2.1 General

(a) Petroleum products are generally a base product such as fuel oil or lube oil with certain additives to enhance performance characteristics. Regardless of how these products are formulated or treated, it is important to prevent or minimize the inclusion of water and solid foreign matter when they are stored, handled, or used. This must be addressed whether or not the use of additives is considered. These additives include cetane improvers, dispersants, detergents, antioxidants, metal deactivators, pour point depressants, wax crystal modifiers, buffers, water dispersants, and biocide. In some cases, additives are included at the factory as specified. In other cases, additives are injected by the railroad or pipeline at the site to control problems specific to their location.

(b) The following are some specific problems associated with diesel fuel handling:

1. Water in the fuel causes many problems. The major one being water serves as a place for bacteria of all types to grow. This bacteria then attacks the petroleum and metal components causing a jelly type mass to develop which can plug fuel filters. In addition, injectors can become stuck, resulting in premature engine failure as well as damage to the injectors themselves.
2. If fuel is unstable, a gummy sludge is formed. This gummy mass becomes insoluble in time and precipitates in tanks and on injector parts, causing them to stick. It can stick to the injector parts when they cool down, causing them to malfunction. This can result in poor combustion and possibly fuel starvation. The overall result is a poorer burned fuel which produces smoke and right of way fire potential through the emission of burning carbon particles through the locomotive stack.
3. Water can dissolve in diesel fuel in very small amounts. As the temperature of fuel decreases, this water is forced out of solution. This water can freeze below 32°F causing iced filters, strainers, and fuel lines. This reduces fuel flow and could result in engine shutdown if bad enough.
4. Wax in diesel fuel can plug fuel filters in cold weather resulting in engine shutdown.
5. Some fuels contain sulfur compounds capable of reacting with the fuel to produce acid products. These "sour fuels" can also contain dissolved gaseous sulfur which is corrosive to iron and steel in fuel storage and delivery systems. This type of fuel should be neutralized with an additive.

5.6.2.2 Low Grade Fuels

Additives can be used to increase the quality of lower grade fuels which can be obtained at a lower cost. It may be cost effective for the railroad to buy a lower grade fuel and upgrade it based upon the railroad's particular needs. The evaluation of additives must be done by people trained in the use of such materials and who are thoroughly familiar with diesel fuels. Only use products of reputable manufacturers.

5.6.2.3 Proper Handling Procedures

If additives are used, additive containers may require diking and other spill control measures. Refer to manufacturer's recommendations on product literature and material safety data sheets for recommendations on proper handling procedures. Dikes and other spill control measures may be applicable.

5.6.3 Recommended Design of Tanks

5.6.3.1 General

(a) It is customary to contract for the fabrication and erection of standpipe-type storage facilities to outside steel companies on foundations furnished by the railway company. It is recommended that the tank itself be constructed in accordance with current API-650 Specifications for Welded Steel Oil Tanks. In addition, tanks should comply with current regulations as outlined in Chapter 13 Article 5.6.1.1 b.

(b) For large installations, above ground cylindrical welded steel storage tanks are generally used. Underground tanks save the cost of dikes and allow the ground above to be used for other purposes such as roadways. Underground tanks are more expensive to install, however. The registration, monitoring of tank systems (i.e. inventories, monitoring wells, etc.), and testing systems (i.e. tank integrity, piping cathodic protection, tank cathodic protection, etc.) are making this option a less desirable option in large installations. For products with high flash points, such as gasoline, fire marshalls may require tanks to be underground.

(c) Adequate storage should be available for reserve in the event of interruption of delivery from normal sources of supply. In the past, it was recommended for small stations to have thirty days of reserve and sixty days reserve for larger stations. This would be cost prohibitive for some of today's centralized fueling facilities. A risk analysis should be performed based on the available sources of supply and the chance for interruption of service to determine the required capacity.

5.6.3.2 Recommended Design Features

(a) Refer to the most current API-650 Specifications for Welded Steel Oil Tanks.

(b) Items to consider:

1. Inlet and outlet connections should be placed 6 to 12 inches from the tank bottom.
2. Water drains should be equipped with a method of collecting the water drawoff. The drawoff should go to a valved pumpable sump so water can be discharged to an oil separator or treatment facility. By using a valve in the sump, oil from a tank failure is prevented from directly entering the wastewater treatment system. By using a manually operated pump and visual inspection of the sump contents, oil entering the treatment system can also be minimized.
3. Manways should be provided for access.
4. Temperature compensated level indicators assist in providing inventory balance.
5. Vents should be designed in accordance with the most recent API Venting Guide.
6. Ladders or steps should be provided to the roof and meet OSHA requirements.
7. An internal check valve should be provided in the outlet pipe to automatically close in the event of fire and when required by state or local fire laws.
8. In the past, the EPA has not regulated No. 2 diesel fuel for vapor recovery, however, some state or local requirements are more stringent than federal requirements and should be checked before installation. There are requirements for products with high vapor pressures like gasoline, however.
9. A high level switch should be provided to stop delivery to the tank or sound an alarm to prevent tank overflow.

5.6.3.3 Tank Foundations

(a) The average standpipe-type tank, full of oil, seldom weighs more than 2,000 pounds per square foot of bearing surface, and in most cases can be installed on relatively inexpensive foundations. At locations where the bearing capacity of the soil is 3,000 pounds per square feet or more, level the site and remove any soft top soil, then install gravel or medium-sized crushed stone over the foundation area to a height of at least 12 inches above finished grade and to a diameter slightly greater than that of the tank. This material should be confined at its perimeter by a circular reinforced concrete curb, which can also serve as support for the outer rim. The surface area between tank shell and curb, if any, should slope outwardly and be paved with concrete or asphalt. As a rust preventative measure, a 3 inch sand cushion, well mixed with a good grade of sulfur-free oil, should be spread over the area on which the bottom of the tank will rest.

(b) The very bottom layer of the tank foundation and dike should be made of an impermeable or impervious material such as an oil resistant PVC or at the minimum, clay, respectively. The liner should be installed on a well prepared base as directed by the manufacturer. This will prevent oil which could leak from the bottom of the storage tank from entering the water table.

(c) For details on dikes see Section 5.6.5.3.

5.6.3.4 Painting of Tanks

(a) All oil storage tanks should be painted to prevent corrosion of the exterior of the tank. The majority of railroads in North America use an aluminum or white paint because this reflects rather than absorbs heat from the sun and tends to prevent fuel degradation during hot weather.

(b) The interior bottom and bottom three feet of the tank should be coated with a rust resistant material which will protect the steel from corrosive residues and water which may accumulate in the bottom of the tank. This coating should also be compatible with oil so the product inside the tank does not degrade the coating.

5.6.3.5 Inspection of Maintenance of Diesel Fuel Storage Tanks

(a) Diesel fuel oil tanks should be inspected at six month intervals, spring and fall, primarily to determine the amount of water and sediment accumulated in the bottoms. This includes all tanks, regardless of size, located above ground or underground. The accumulation of water and sediment can ordinarily be removed from above ground tanks by gravity through valved outlets near the tank bottom. If there are no such outlets, water and some sediment can be removed by pumping through the opening in the top of the tank.

(b) Fuel samples should be taken by a qualified inspector using an ASTM "Tank Thief" or bacon bomb. A tank thief is sampling device having a cast aluminum case 2 or 4 inches in diameter by 12 inches long, with a spring actuated butterfly valve at the bottom. It is open at both ends when being lowered into the oil, and the butterfly valve can be closed at any level to take a sample; it closes automatically 1/8 inch above the bottom. A bacon bomb is usually made out of stainless steel and comes in various diameter sizes, the most common being 2 inches. It has a normally closed plunger which opens when it hits the bottom of the tank drawing a sample. A separate wire actuator can be used to raise the plunger at any desired depth in the tank to draw a sample. Samples should be taken at the half-full level and near the bottom for analysis and report. The report should include the pH value and the salinity of the water, as in a few cases it has been found that the water was acid or corrosive in nature and may cause the tank bottoms to corrode. Accumulation of water and sediment over a six month period usually is negligible. A substantial accumulation of water indicates ground or surface water leakage into the tank, or poor product. Above ground tanks are less subject to such leakage and less expensive to maintain.

(c) Fire hazards associated with working in and around tanks used for storage of diesel fuel oil is generally recognized especially in confined and poorly ventilated spaces. If tanks are emptied for inspection, cleaning, or repairs, the interiors should be thoroughly cleared of fumes by steaming or venting and then tested for available oxygen with a Lower Explosive Limit (LEL/O_2) meter. Men entering tanks must be equipped with appropriate respiratory protection and an appropriate means of rescue. OSHA regulations on confined spaces may apply.

(d) Inspection of tanks for general condition and determination of repairs needed should be made annually by qualified personnel. The inspection should include visual inspection for tank pitting and/or a leak check by level measurement or other method. A more detailed inspection of welds may be needed on older tanks. Acoustic, pressure testing, ultrasonic and other emerging technologies should be considered. Refer to the most current version of the API—"Guide for Inspection of Refinery Equipment" Chapter 13 "Atmospheric and Low Pressure Storage Tanks."

(e) Service piping from tanks should be arranged so the entire depth of fuel can be used or circulated to prevent aging which results in oxidation or enlargement of particle size which would cause filter plugging or locomotive fuel injector trouble. If the tank drain will not remove all water and sediment, a suitable inhibitor should be used to protect against corrosion of the tank bottom.

(f) In addition to semi-annual and annual inspections of tanks to determine general conditions and need of repairs, the following should be checked:

1. Tanks and their surrounding areas should be clean and free of weeds, grass, paper, and other combustible material. There should be good drainage away from tank to minimize corrosion.

   Grass and other vegetation should be removed from the area. It is preferable to apply a soil sterilizing formulation within the fire wall at frequent intervals to prevent regrowth.

2. Air inlet screens must be clear of dirt, rust, or frost, as it is possible to rupture or collapse a tank by pumping oil into or out of it if the vents are frozen or otherwise clogged.

3. All valves, manual or automatic, provided for normal operation or emergency protection should be opened and closed several times to be sure that they are in operating condition.

4. Diesel fuel oil storage tanks should be wire grounded, and such grounds should be thoroughly checked.

5. Storage tanks supported on grade should be cathodically protected, with protection tested at recommended intervals.

5.6.4 Oil Receiving and Distribution Facilities

5.6.4.1 General

(a) Oil delivery may be made by tank cars, highway truck transports, barges, private pipelines, common carrier pipelines, or combinations of these methods, depending upon the locality, economics, etc. Planning should include the possibility of using all available means of delivery and designing the facility so that the method of delivery can be varied in the direction of greatest economical advantage without interruption of service.

(b) At locations where tank cars, highway transports, and barges are unloaded, connections have to be made to on site piping. These connections are areas of potential leakage or spillage. Provisions should be made to provide these areas with spill collection facilities such as concrete pads or track pans with drainage to an oil separator. Supplies of absorbents should be readily accessible for spills of lesser magnitude.

5.6.4.2 Oil Receiving Facilities

(a) Oil receiving facilities should include provisions to pump the product into on site storage tanks, if required, as well as metering facilities for inventory control. The value of inventory control is two fold. First, it maintains economic integrity on fuel purchases. Second, it provides an early warning system for fuel leakage.

(b) Oil receiving can be done from tank trucks, tank cars, barges, or pipelines. An loading facility should include or consider the following:

1. A quick disconnect fitting so the tank truck or barge can be connected to the facility piping.

2. A check valve to prevent spillage from the pipe onto the collection facility.

3. A mesh strainer should be installed on the suction side of the pump to prevent foreign matter from entering the pump or meter.

4. A permanent transfer pump is recommended especially when receiving product by tank truck since on board truck pumps can cause the incoming fuel meter to read high. The transfer pump should be designed to accommodate the size of the fuel meter as well as to prevent unnecessary delay of trucks.

5. An air elimination system to prevent the fuel meter from reading air which results in inaccurate inventory control. The air eliminator should be vented to the outside area which has drainage to the oil separtator system.

6. A temperature compensated fuel meter to assist in inventory control. Piping should be included so the fuel meter can be calibrated periodically.

7. A printer should be provided outside the pump house so each load can be recorded. All other equipment except the piping connection should be inside a locked building to prevent tampering. Telemetry equipment is available to transmit this data to a central inventory control point.

8. Filtration facilities should be considered to protect stored oil from contamination with a dirty fuel consignment. Preliminary filtration should be standard practice as oil is delivered into storage. Alternately, filtration can be done at the locomotive fueling facility instead.

   (a) Filters should be replaced when fuel delivery is too low to maintain the fueling schedule and pressure loss approaches manufacturer's recommended limits.

   (b) A drain system for the filter bowl should be designed to permit draining the filter bowl and replacing the elements without spillage upon floor or ground to cause pollution.

   (c) Used filters should be collected and stored in leakproof containers for disposal by a licensed contractor or other approved disposal method. Disposal must meet federal, state, and local regulatory requirements.

   (d) A dual fuel filtering system may be installed to permit continuity of fueling while filter replacements are being made.

   (e) Filters should be located on the discharge side of the pump.

9. Electrical Facilities. The electrical work required for a fuel oil pumping plant consists mainly of power supply to the building, circuit breakers, starters, and a start/stop control system for the pumps. Explosion proof motors and appurtenances should be installed where INFAMY Class 1, Group D materials are handled. Electrical specifications depend upon the material being handled and the environment in which the equipment is located. The state and local building codes will dictate the appropriate requirements.

   A flow control switch in conjunction with a timer limit switch will in due time stop the pump if personnel fail to do so. Start/stop control requirements vary according to local conditions. In general, automatic operation is desirable to the extent the operation can be made fail safe. Recommended types of control are shown under Art. 5.6.4.3.d.

10. A bottom sediment and water detection (BS & W) device can be used to detect dirty or wet fuel. These devices can be set to shut off delivery when such a condition is detected.

11. Provision for additive addition should be made at the suction side of the pump. This allows the pump to mix the additive and provides for introducing treated fuel into the storage tank.

5.6.4.3 Oil Dispensing Facilities

(a) Dispensing facilities for diesel fueling consists of piping from the fuel storage tank to a pump house, a pump facility, distribution lines to the dispensing location (service track), metering, if desired, hoses and nozzles to connect to the locomotive day tanks required for remote fueling locations, and controls to keep the day tanks full and from overflowing. Secondary filtration is often provided as a part of these facilities. Today's high speed fueling locations should have adequate provisions for collecting spillage at the service track. In general, the same principles apply to dispensing facilities as apply to receiving facilities. Proper metering can provide information necessary to track fuel leaks and even be used in energy conservation studies if meters are placed at each nozzle.

(b) Employees fueling locomotives should be adequately indoctrinated in their jobs with complete understanding of automatic fueling and should be held responsible for any spillage. There should be a means to detect spillage at the end of each shift, and proper action taken immediately as to the cause. Proper supervision can eliminate most fuel oil spillage. If the above practice is strictly adhered to, the fuel platform will be a clean, safe place to work. Personnel will take pride in seeing that their performance meets the satisfaction of the management.

The USEPA requires petroleum spills be reported to the National Response Center or EPA and the appropriate state agency. These reporting requirements should be covered in the SPCC Plan referred to in Section 5.6.1.2.b.2.

(c) Fuel Outlets

1. Spacing should be provided to handle various classes of locomotives. Counterbalance type arrangements provide easier handling of the final hose connection and make it safer to handle. The length of and number of outlets at the fueling facility should consider the total number of units to be fueled on one track at one time.
2. A quick opening quarter turn plug type valve should be installed between the header and pressure reducing valve for emergency shut off in case of hose rupture or nozzle and coupling coming loose or failing to close. The valve should also have internal pressure relief to operate at slightly higher than normal header pressure to relieve any locked pressure build up between valve and closed nozzle due to locked in line pressure and fuel expansion due to temperature rise.
3. Final delivery of fuel oil to locomotives is made using 2 to 2-1/2 inch ID hose, maximum working pressure between 100 and 125 psi. Hose should be no longer than necessary with proper counterbalance arrangement. This will assist the employee in maneuvering the hose in making the quarter turn connection to the locomotive in a safe and efficient manner.
4. Automatic shut off nozzles should be used in all fueling operations including normal or low flow as well as high speed. Adequate protective devices should be included in the design to prevent nozzle damage through rough handling.

(d) Fuel Cranes

1. Crane Masts are made up of a 4 inch riser pipe 10 to 12 feet high with a swivel joint and short horizontal extension of 2 or 1-1/2 inch pipe at the top and a drop hose to make the final connection. Swivel joints provide operating flexibility, and the working range is adjusted by the amount of hose used.
2. Vertical Swing Masts have a double swing joint and vertical (pulldown) extension pipe at top of riser, similar to an overhead (dome) unloading connection except that the hose takes the place of the suction drop pipe. Although more expensive, this type of mast provides greater working range with less hose, occupies less space, and has other operating advantages over the crane type mast.
3. Hose outlets on the bottom of overhead manifolds should be spaced as required to permit the use of relatively short hoses in fueling locomotives. Dummy fueling connections can be placed on manifold supports to hold nozzle out of the dirt and to prevent it from damage when not in use. Receptacles can include a drain directly to the oil separation system to collect oil dripping from leaky nozzles.

(e) Pump and Pump Control

1. Bypass piping for high speed fueling. A bypass arrangement is usually needed for an installation where multiple diesel units must be filled quickly and concurrently. The purpose of the bypass is to maintain the lowest pipeline pressure consistent with high speed fuel delivery and to minimize the shock as fuel nozzles are suddenly closed at the fueling platform. In addition, it provides a minimum flow through the pump to prevent it from overheating. A relief valve is located on the discharge side of the pump on the inlet side of the filter and is set to maintain a sufficient pressure in the header at the fueling area to permit full fuel delivery to the most remote fuel outlet. Fueling

pumps are normally started prior to fueling operations and precautions should be taken to prevent the pump and fuel from heating by friction during this no delivery period. It is preferable that the bypass discharge should extend back to the storage tank, but in some instances it may be advantageous to return it only as far as the pump suction. The bypass valve should be hydraulically operated, pilot controlled, modulated type, activated by line pressure through a pilot control system, opening fast to maintain steady line pressure and closing gradually as locomotive fuel outlets are opened, to prevent high pressure surges in the pump system. Bypass piping design should also include specific measures to prevent hydraulic hammer in the system.

2. Recommended practice is to provide a push button start-stop switch on each fueling mast with some kind of red light indicator to furnish additional visual evidence of its position. An alternate to the start/stop switch is a mercury tube switch installed on the lever arm of the loading valve which will automatically stop pump operation when the valve is closed.

3. A timer, pressure switch, or flow switch can be used to automatically stop the pump.

5.6.4.4 Oil Distribution Systems (Piping)

(a) This section covers the piping from receiving facilities to storage to outbound fueling up to the connection to the locomotive. Generally, oil distribution piping can be installed above or below ground. Pipe installed above ground is generally more acceptable because leaks can be easily spotted. All underground piping should be protected against corrosion as covered in d. below and in Part 6—Corrosion Control. In addition, periodic pressure testing of underground lines should be done in addition to inventory balancing to detect underground leaks.

(b) Pipe Size and Class

1. Diesel locomotive fuel tanks range from less than 3,000 gallons to the 20,000 gallon fuel tender cars. Piping should be sized so the time required to fuel is at an acceptable level. Multiple fuel nozzles on one header and fueling rates from 200 to 300 gpm per locomotive should be considered.

2. Pressures of 50 to 75 psi are generally sufficient to provide adequate flow. Pressure reducing valves may be required to reduce the pressure at the service track to the rated pressure of the fueling nozzle.

3. Welded joints are preferable for above ground steel piping.

4. While welded steel pipe is the preferable choice for both above and below ground piping, other pipe such as cast iron, suitable plastic, or steel pipe with extruded coatings of PVC is also used. It should be sized to hold friction losses as low as practicable. Joints may be welded, flanged, screwed, mechanical joint type with bolts, or press type using neoprene or suitable gaskets.

(c) Depth of Bury for Underground Pipe and Use of Casing

1. Depth of bury for underground pipe and use of casing are covered under Section 5.1—Specifications for pipelines conveying flammable substances, contained in Part 5, Chapter 1 of the AREA Manual.

(d) Protection Against Corrosion and Leakage

1. Leaks in underground fuel oil lines are difficult to detect and can result in considerable product loss, soil and groundwater contamination, and subsequent expensive pollution problems. Regulations require that underground piping be coated with an anti-corrosive preservative and to wrap the pipe with tarred or plastic wrapping either before or during installation. Additional precautions are to backfill around the pipe with sand, use cathodic protection, as specified in Part 6—Corrosion Control, or nonrusting pipe, i.e., plastic. Pressure testing similar to water line testing covered by AWWA specifications is recommended. Regulation 40 CFR Part 280, Underground Storage Tank, applies.

(e) Installation of Lines Above Ground

1. Supports for above ground piping should be spaced as necessary to provide sufficient structural support and may be constructed of rail, reinforced concrete, or other support members as applicable. The total expansion/contraction due to changes in temperature is not usually significant and may be provided for by the conventional methods, such as loops, swing joints, and expansion joints. Thermal expansion can be a problem in above ground piping especially when the fluid is trapped between two closed valves. If this is the case, a pressure relief valve is recommended to alleviate this.

5.6.5 Secondary Containment and Spill Prevention

5.6.5.1 General

As recommended practice, there should be methods available for containing spills of petroleum products. Some of these methods will be incorporated into the design such as track pans, dikes, automatic shut-off devices, etc. Other considerations include absorbents, spill containment material stored at the site, and monitoring wells to be used after a spill occurs.

5.6.5.2 Inventory Control

(a) Proper inventory control can go a long way toward preventing spills in the first place. Petroleum products are valuable. Fuel oil has ranged from $0.40 to $1.00 per gallon in the last decade so reduced spillage means dollars saved. By monitoring fuel received, fuel on hand, and fuel dispensed, an inventory control system can be established so that fuel discrepancies can be investigated immediately. These systems can be equipped with readouts that can be read at the site. Telemetry is available so this informaiton can be read at the local yard office or even the railroad's central office building. Discrepancies greater than a certain preset limit should be investigated. In addition, discrepancies which indicate numerous shortages in a row should be investigated. Preventing spillage saves costly environmental clean up and could prevent fines from being levied.

(b) The metering equipment should be inspected regularly to assure proper operation. The meters should be calibrated periodically (once or twice a year) depending on the amount of product received. Outside expertise should be considered for this calibration especially if the railroad wants to use the information to dispute delivery receipts from their suppliers.

5.6.5.3 Dikes

(a) Storage tanks must be surrounded by dikes designed to contain the contents of the tanks and any undrained stormwater within the dike. Dikes should be made out of impermeable material such as clay, concrete, or plastic lining beneath a soil layer. Generally, earth dikes are used where space is readily available. Earth dikes are generally 4 to 6 feet high with a 3 foot crown and a 1-1/2:1 to 3:1 foot slope depending on the state. The dike should be capable of containing 100% of the volume including rainwater of the single largest tank within the diked area for individual tanks. The dike should be capable of containing 100%, including provisions for rainwater, of the total of all tanks within the diked area when those tanks are connected with common piping. Where space is limited, concrete or block dikes are more desirable. When space is severely restricted, a separate steel tank wall can be constructed around the tank as a dike. This would be similar to a double walled under ground storage tank. State and local agencies referred to in Section 5.6.1.1 b have guidelines and/or regulations on dike volume, crown width, degree of imperviousness, distance from tank to dike, etc.

(b) As mentioned in Part 5.6.3.2 b 2, a water drain should be included in the dike. If the dike is designed to hold oil, it will also retain rain water. This water often becomes contaminated with oil present in the dike, from draining water off the tank, and small spills. A sump should be installed to drain this water to an oil separator. Note, a normally closed and locked valve or pump should be used to prevent uncontrolled discharge from the dike. Only manual controls, not automatic, should be be used to control the pump in the dike. This will prevent pumping oil accidently into the separator if a tank did fail.

5.6.5.4 Secondary Containment

(a) Facilities Design

1. Various collection methods exist to collect spillage of petroleum products. These systems include track pans, concrete pads, geotextiles, etc. These systems are discussed in more detail in Chapter 13 Part 1.2.5.5.

(b) Spill Control

1. When a spill occurs which is not contained by the facilities designed to do so at the site, it will be necessary to implement the previously prepared Spill Prevention Control and Countermeasure (SPCC) plan as outlined in 40 CFR Part 112 EPA Regulations on Oil Pollution Prevention or other contingency plans as required. People who work at petroleum handling facilities should be familiar with this plan and keep it at a designated location at all times. The SPCC plan should be revised when changes are made which affect the plan. For instance, a new response coordinator may be needed when personnel at the site change. EPA regulation 40 CFR Part 112 requires that the SPCC plan be updated at least every three years. The SPCC plan should include reporting requirements to be followed to comply with federal, state, and local regulations.
2. Spill containment equipment, such as oil absorbent pads and booms to stop spills once they reach a stream, should be readily accessible as a matter of recommended practice. Personnel should be instructed in the use of this equipment and likely locations for deployment. When spills of this nature occur, it will be necessary to notify the National Response Center or EPA, and the appropriate state agency.

Proposed 1990 Manual Revisions To Chapter 14—Yards and Terminals

Parts 3, 4 and 7 of Chapter 14 are proposed to be revised:

1. Part 3—Freight Delivery and Transfer and Part 4—Specialized Freight Terminals

It is proposed to consolidate Parts 3 and 4 by incorporating Section 3.2 on Team Yards in Part 4 as Article 4.6.1.2 (c). Section 3.1 on Freight Houses and Section 3.3 on Driveways will be deleted, but references will be made to related material in the Manual and Proceedings. Changes are recommended for track centers in team yards and additional Part 4 material for Bulk-Solids is proposed as Section 4.4.2. The specific changes are:

Part 3. Foreword (Page 14-3-1)

The Foreword will be reworded to:

FOREWORD

This part deals with the engineering aspects of freight (commodities, merchandise, etc.) handling at the points of origin and delivery, in carload lot or less-than-carload lot, or in the consolidation of LCL freight from a greater to a lesser number of cars, or vice versa, or where it it desired to transfer packaged freight from foreign line cars into home line cars but more specifically from railroad freight cars to trucks for forwarding to destination. The facilities are essentially supplementary and/or alternative units and should be so located, designed, and operated in relation to each other and to the lines tributary to them, as to give the most economical results for the railway as a whole.

The specifics to establish, construct and maintain such facilities are included in relative section of this Manual as follows:

Freight Houses	Page 14-4-5 Section	 4.2.3.7
Team Yards	Page 14-4-15 Section	 4.6.1.2c
Driveways	Page 14-4-4.0 Section	 4.2.3.3

Complete design criteria and additional pertinent information are readily available in Bulletin Vol. 77 (1976) page 87 and Vol. 83 (1982) page 363.

Article 3.2 Team Yards (Page 14-3-5)

Article 3.2 will become Article 4.6.1.2 (c) in Part 4 and be worded as follows:

(c) The location of a team yard should be such that it will be convenient for use by shippers and consignees, and also as convenient as possible to a freight house, so that the receipt and shipment of freight may be easily under control of the freight agent's force.

(1) Equipment

A crane for handling heavy freight should be provided when required. A motor truck scale, with office, should be provided near the main entrance to the team yard when required.

(2) Tracks

Switching tracks for holding and working cars should be provided in the immediate vicinity of the team tracks and so arranged as to facilitate the switching of these tracks.

The spacing of tracks, where multiple team tracks are built, may be fixed by regulatory bodies, but it is recommended that the minimum distance between track centers be 14 ft.

The distance between track centers where the driveway is located between tracks should be 16 ft. greater than the width of the driveway.

Article 4.6.1.2 Functions of Railway and Marketing Facilities (Page 14-4-15)

Change first paragraph to read:

A produce terminal should be considered to include (a) railway facilities and (b) wholesale marketing facilities (c) team yards.

Article 4.6.1.3.1 Track (Page 14-4-16)

Change subarticle (e) to:

(e) Track centers should be not less than 14 feet

4.4 Bulk Solids

Additional Part 4 material for Bulk-Solids is proposed as Section 4.4.2. This material is a condensation of the study previously published in AREA Bulletin 713, Volume 88, December 1987 on "Bulk Material Handling Systems for Granular Solids," and is titled Design of Bulk Granular Solids Terminals.

The new material addresses Terminals for dry bulk solids such as aggregates, coal, ore, coke, etc. which suffer little or no damage from it's normal handling or exposure. The revision incorporates many features relating to unit train technology, which are applicable to the above commodities. The new Section 4.2.2 is as follows:

4.4.2 Design of Bulk Granular Solids Terminals

4.4.2.1 Introduction

Railroad bulk granular solids terminals are specialized freight terminals which are used to transfer bulk lading from point of origination to rail cars, transfers from rail cars to point of destination or to transfer bulk lading between rail and other modes. For additional pertinent information regarding design of bulk granular solids terminals, refer to Bulletin 660, Proceedings Volume 78, November-December 1976, pages 255-259, and Bulletin 713, Proceedings Volume 88, December 1987, pages 395-424. This section is applicable to dry bulk solids such as aggregates, coal, phosphates, ore, and coke, which suffer little or no damage from normal handling or from exposure.

4.4.2.1.1 General

Factors affecting terminal design include number and types of materials to be handled, the size of shipment (unit train, ship or barge, multiple car, single car), the physical characteristics of the site, and the degree of processing and storage to be done on the site.

4.4.2.1.2 Site Selection

The site should be selected to accommodate both near and long term development of the site. The following factors should be considered during layout and planning of the site.

4.4.2.1.2.1 Environment

Chapter 13, Environmental Engineering, AREA Manual for Railway Engineering discusses environmental considerations in detail. Environment factors to be considered include:

1. Air, especially dust control and collection
2. Water
3. Noise and lighting
4. Rainfall runoff
5. Archaeological and Historical Sites
6. Adjacent development
7. Proximity to residential areas
8. Avoidance of wetlands and other ecologically sensitive areas

4.4.2.1.2.2 Economics

Site selection and arrangement should allow for economy in movement of materials and transportation operations.

4.4.2.1.2.3 Size

The terminal should include sufficient land area to accommodate the ultimate development of the terminal. Sizing of equipment and structures should allow for expansion and flexibility.

4.4.2.1.2.4 Access

Design of highway access should consider the requirements for service vehicles and heavily loaded trucks normally associated with bulk terminals. Water access should include sufficient draft maneuvering space and berthing for the size and type of vessel anticipated at the terminal.

Rail access should be designed for the longest and heaviest anticipated shipments arriving or departing by rail at the terminal.

4.4.2.1.2.5 Utilities

Siting and planning should account for required utilities at the site. Water will be required for fire protection, dust control, and washdown. Bulk handling equipment may require a significant amount of electrical power, and some sewage disposal is generally necessary.

4.4.2.1.2.6 Zoning

Siting near residential, commercial, or recreational areas is almost always a controversial issue, with potential delays from hearings and other legal process. Many governmental entities have enacted zoning laws that govern construction of these facilities. It is desirable to avoid areas where rezoning is necessary in order to construct the facility.

4.4.2.2 Facilities

Facilities vary with the size and function of the terminal, but can be classified into the following categories.

1. Unloading
2. Loading
3. Storage and Reclaim
4. Conveyance
5. Sampling and Weighing
6. Service Facilities

4.4.2.2.1 Unloading Facilities

Unloading facilities vary from low volume single or multiple-car systems to the high-volume terminals capable of unloading unit trains. The unit train terminals may be rotary or bottom dump unloading. In some specialized applications, side-dump equipment or trains equipped with under-hopper conveyors may be more economically utilized, which simplify or eliminate fixed unloading facilities.

4.4.2.2.1.1 Unloading Facilities for Single or Small Multiple-Car Shipments

A small under-track pit with lading removal by conveyor or small mobile equipment is economical in many cases for small shipments. Design of a pit of this type is shown in Section 8.4 of this manual. Car shakers may be used to aid in clearing lading from the car.

Rotary dumpers can also be used for multiple-car shipments. Rotary dumping may offer advantages if lading tends to freeze or to clog during unloading from bottom-dump equipment. Rotary dumpers can be equipped to take various sizes of cars when the cars are uncoupled before dumping.

4.4.2.2.1.2 Unit Training Unloading

Unit train unloading facilities must be designed to unload as quickly as possible to take advantage of the high utilization economies of unit trains. Unloading without uncoupling or switching should also be a high priority in most instances.

Fast unloading automatic bottom dump hopper cars unloading in-motion over a pit or trestle provides the fastest unloading time of any bulk granular material unloading system. The unloading facilities must therefore be designed for high mass flows through the facility. Particular attention should be directed to prevention of "bridging" of material in the bottom of the unloading pit, and reliability of feeders and conveyors to minimize expensive delays in train unloading.

Rotary dump facilities using train positioners and rotary coupler equipped cars also have a high unloading rate. Rotary dumper systems generally utilize cars with lower tare weights, and thus higher capacities per car. Reliability of the dumper machinery is a key item in the design of rotary dump unit train facilities to minimize expensive delays in train unloading. Train positioner design should provide sufficient power to handle the unit train easily during unloading.

4.4.2.2.1.3 Frozen Lading

To aid the unloading of frozen lading in cold weather climates, provisions should be made for thawing lading which is susceptible to freezing. Additives to the lading, thaw sheds or pits, and car shakers are all methods which have been used to aid the unloading of frozen lading.

4.4.2.2.2 Loading Facilities

For all loading facilities, accurate weighing of lading is essential to take full advantage of the car capacity without overloading.

Loading of small shipments may be accomplished with clamshell-equipped cranes, shovels, or other mobile equipment. Hoppers over the track, or conveyors can also be used to load single or small multiple car shipments economically.

For unit trains, a high capacity load-in-motion system should be considered to minimize train delays. Weighing bins to accurately meter lading into each car, based on the car's empty weight, will allow maximum advantage of the capacity of the train.

In general, excess switching should be minimized during loading operations.

4.4.2.3 Storage and Reclaim

Most bulk granular materials handling systems require some type of storage and reclaim system. Storage may be necessary for seasonal or environmental reasons, surges in material flow, blending, or through put demands. Multiple types of materials may be stacked, and blending of materials may be required.

Design of storage and reclaim systems should take advantage of gravity flow wherever possible, and also minimize handling of materials. Stacker/reclaimer equipment should be sized for the expected terminal throughput, so that train loading and unloading are not limited by this equipment. Covered stacks, silos, or sheds should be utilized to protect materials from exposure if necessary. Dusty materials should be covered or otherwise protected to control fugitive dust emissions.

4.4.2.4 Conveyance

Conveyors are the links between the functions of most bulk terminals. Conveyor design should consider the desired material flow rate, material density, and angle of repose. If the terminal may expand at some future date, provisions for dual conveyors or other necessary facilities should be included in the initial design. Some conveyors may be critical enough to require dual systems so that delays in loading or unloading trains, ships, or vehicles may be avoided.

4.4.2.5 Sampling and Weighing

Sampling and weighing provisions should be provided where required for each material handled. The *AAR Scale Handbook* provides guidance on weighing facilities.

4.4.2.6 Terminal Tracks

Bulk handling terminals will very likely be served by 100-ton cars, with a trend toward 120-ton or 125-ton cars in future years. Trackage and subgrade in bulk handling terminals should be designed for these heavy axle loadings.

For rotary dump installations, heavy anchorage of rail should be used on the inbound side of the dumper to prevent rail creepage into the dumper platen or other machinery.

Rail lubricators may be beneficial for reduction of rail and flange wear.

4.4.2.6.1 Track Geometry—Unit Train Facility

For unit train trackage, consideration should be given to train action and conservative curvature and gradient standards. The following criteria are recommended for unit train trackage design.

• Maximum recommended curvature on loops and lead tracks.	7°-30′
• Maximum recommended gradient on approach to loop.	1 percent, compensated for curvature, 0%

• Gradient on unloading loop.	(level), or slight upgrade to unloading trestle or pit.
• Minimum recommended size loop and lead turnouts.	Number 10.
• Maximum recommended rate of change for vertical curves.	0.12 per 100′ station in sags. 0.20 per 100′ station in summits.

The AAR Train Performance Calculator or other similar programs can be run to verify train performance over the new design.

Trackage affecting the operation of dumpers, train positioners and other material handling equipment should take into account the requirements and recommendations of the equipment manufacturers.

4.4.2.6.2 Track Geometry—Non-Unit Train System

Trackage of non unit-train terminals varies widely with the type of terminal. Switching is more common in this type of terminal, and trackage should be designed accordingly. Yard design criteria from other sections in this chapter should be considered in trackage design.

Sufficient car storage track should be available to accommodate surges in traffic with adequate room allowed for switching cars through the loading or unloading facilities.

In general, the following track design standards are recommended.

- Maximum recommended curvature—12 degrees, 30 minutes
- Minimum recommended turnout—Number 8
- Lead Tracks—as long as the longest storage track
- Gradients—preferably flat, unless cars are moved through the dumper by gravity. Grades for storage tracks should be such that the application of hand brakes is not necessary.

In some terminals, where cars are moved by a combination of gravity and barneys or other car movers, the principles of hump yard design and automatic classification yard design can be used to design the trackage. These principles are found in Part 2 of Chapter 14 of the *Manual*.

4.4.2.7 Structures

Structures in granular bulk material handling terminals should be designed for durability and easy cleaning. Electrical equipment and other sensitive equipment may require air conditioning and dust-free environment. Clearances for railroad and mobile equipment should be considered. Additional guidance for structural design and construction are found in the Manual, Chapter 5, Track; Chapter 6, Buildings; Chapter 8, Concrete Structures and Foundations; and Chapter 15, Steel Structures.

4.4.2.8 Maintenance and Housekeeping

Bulk materials facilities should be designed for easy maintenance and housekeeping. Areas where potential material spills are likely should provide access for loaders and trucks for easy removal of spills. Particular attention should be given to the prevention of dust accumulation. Materials used in construction of a bulk material terminal should be resistant to deterioration from the materials handled.

2. Part 7—Other Yards and Terminal Facilities

The present Manual Section 7.3 is a general notation on Reclamation. This annotation is inadequate to cover present operational procedures and it is proposed to replace existing material with the attached in depth design criteria.

This proposed manual material covers plant(s) to be operated by the Engineering, Stores and/or Purchases & Materials Department, depending on the individual corporation policies. The facility herein defined could include manufacturing as well as rehabilitation of components i.e., frog shop, insulated glued joints, various turnout materials, bridge fabrication shop, girder rehabilitations, track and or turnout panel fabrication, signal shop, etc., The proposed Section 7.3 follows:

7.3 DESIGN OF ROADWAY MATERIAL RECLAMATION AND FABRICATION FACILITIES

7.3.1 Introduction

Roadway Material Reclamation and Fabrication Facilities, hereafter referred to as Plant(s), may be operated by the Purchases & Materials Department, the Stores Department or the Engineering Department. Plants operated by the Engineering Department handle only engineering and maintenance-of-way materials. As defined herein, a Plant may include a facility to sort and classify used other track material (OTM); a Frog Shop to manufacture and rehabilitate turnout components and fabricate insulated glued joints; a Bridge Fabrication Shop to fabricate new girders and rehabilitate used girders; a Turnout Panel Fabrication Facility and a Track Panel Fabrication Facility. A plant may include a Signal Shop but such a facility is not included in this report.

7.3.2 Roadway Material Reclamation Plant

7.3.2.1 Material Sorting

After laying rail, most railroads pick up the old OTM using a work train having a crane equipped with a magnet. The unsorted OTM is put into a railcar. This method speeds up the removal of the OTM from the right-of-way and reduces train delays. The unsorted material is then taken to a Roadway Material Reclamation Plant for sorting and classifying. Another method of reclaiming OTM is hand sorting the material at the site and placing it into piles to be picked up later. This method is very labor intensive and requires more track time. For considerations in handling of OTM refer to "Economics of Methods to Dispose of Scrap and Obsolete Material" in AREA Bulletin 722, Vol. 90 (1989).

7.3.2.2 Material Handling Facilities

The Plant should be provided with tracks and hard surface driveways for the efficient handling of materials by rail and by truck.

Incoming OTM for the Plant is unloaded along the track adjacent to the Plant with a crane equipped with a magnet. The crane can be rail or rubber tire mounted. The material can be placed directly into a hopper for sorting or it can be placed on the ground for sorting later. In sorting the OTM the material is fed from the hopper to a conveyor belt for classification. The usable second-hand material is usually moved into the storage area with forklift vehicles. Other vehicles used for intraplant transfer of OTM include trucks and railcars.

When unsorted OTM is to be delivered to the Plant in railcars, it is important to design the Plant so the cars can be unloaded immediately and released for revenue service or for loading outbound shipments. The Plant layout should be such that moving the material to a different location or double handling will not be necessary prior to sorting and classification. This can be accomplished with an elevated conveyor belt mounted on a gantry. Previously unloaded OTM is picked up with a magnet on a crane mounted on the gantry. The OTM is loaded into a hopper that feeds it onto a conveyor belt. Workers along side the conveyor belt, who are assigned specific items to inspect, remove reusable items and deposit them into another hopper adjacent to the belt. The material deposited into the hopper falls into a container or railcar at ground level below the hopper. The container when filled can be moved to storage with a forklift. Scrap material left on the belt is deposited in a railcar or other container for shipment to a scrap dealer. A railcar containing usable materials is either shipped out to be used for construction or unloaded for storage.

Plants may be movable or stationary. A stationary Plant has a lower first cost but usually requires double handling, moving of materials several times, or storage in railcars until the materials are sorted and classified. This system is similar to the plant described above but is fixed in one location.

Other layouts include conveyor belts near the ground with workers removing assigned items and putting them in piles on the ground. Any items not removed are classified as scrap and dumped in a railcar or other container.

The Plant layout, the flow of materials and the amount of automation should be determined by good industrial engineering principles as specified in AREA Manual for Railway Engineering Chapter 16, Part 6, Railway Application of Industrial Engineering. Studies should be made to determine to what extent automation is justified. Identification of materials by the use of Closed Circuit Television and pushbutton controls to remove materials from the belt should be considered, but used only when the cost of such an installation can be justified.

The materials normally sorted and classified in a Roadway Material Reclamation Plant are joint bars, compromise joints, tie plates, rail anchors, spikes, nuts and bolts, and other small track material. Since the value of reclaimed material usually exceeds the cost of sorting and classifying the material, most railroads have facilities to perform this work. Often new material gets mixed in with the used material during track relays which makes the reclamation operation even more profitable.

Some railroads straighten bent spikes, while others scrap all second-hand spikes. Spike straightening machines are available, but a cost analysis should be made to justify spike straightening before this operation is included in the Plant.

Nearly all railroads recrimp rail anchors so that they are tight when reapplied.

Most tie plates are reusable as is, but they may be repunched to be used with different rail sections or spiking pattern.

Most bolts are reusable after rethreading them. A new nut and lock washer are usually used on rethreaded bolts.

7.3.3 Frog Shop

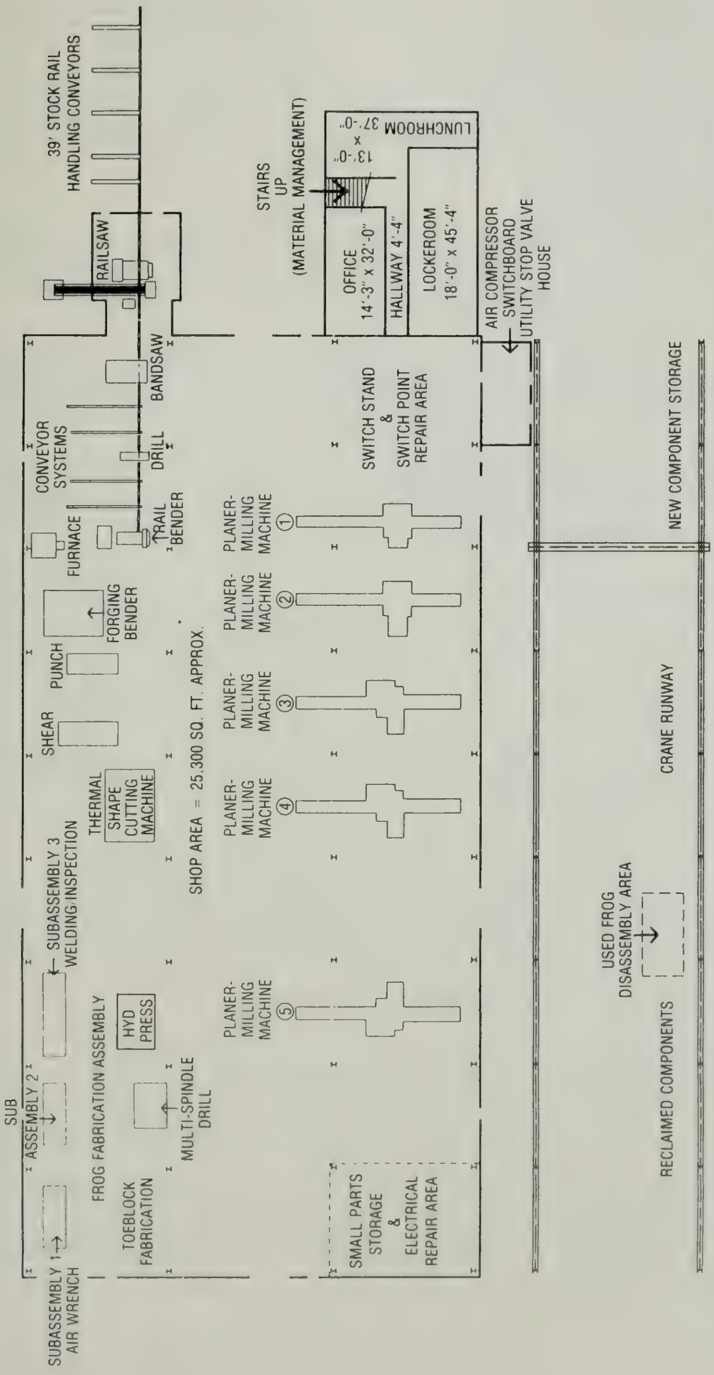
The Frog Shop receives rail, steel sections, and other heavy material, usually in railcars. Rail is used at the Frog Shop for the manufacture of frogs, guard rails, switch points and stock rails. The fabrication of glued insulated joints also requires pieces of rail, usually new. Rail is normally unloaded from railcars with an overhead crane and placed on racks equipped with rollers for easy movement of the rail into the shop area. Various materials are moved inside the shop with conveyor belts or rollers and jib cranes are usually used to lift heavy items. Forklift vehicles are used to move the completed products out of the Frog Shop to load them or place them in storage.

Other activities done in the Frog Shop include manufacturing of machine bolts and drift pins, fabricating of slip joints and scale rails, explosive hardening of frogs and glueing of insulated joints.

A typical layout for a Frog Shop is shown in Figure 1.

7.3.3.1 Manufacturing New Turnout Components

Turnout components to be manufactured include switch points, stock rails, heel blocks, switch rods and clips, gauge plates, railbound frogs and guard rails. Equipment used in a Frog Shop to fabricate



TYPICAL LAYOUT
FROG SHOP
FIGURE 1.

turnout components includes lathes, planes, punches, drills, overhead cranes, jib cranes and conveyors or rollers to move the rail and other components through and in and out of the shop.

7.3.3.2 Rebuilding Turnout Components

Rebuilding frogs, guard rails, switches, switch stands and crossing diamonds should take place at or near the Frog Shop, since some of the machinery and supplies needed to manufacture these items are also required to rebuild them.

7.3.4 Bridge Fabrication Shop

The Bridge Fabrication Shop receives girders and steel sections, usually by rail. Other incoming materials, such as bolts, can be shipped either by rail or truck. An overhead crane or gantry is used to move materials into, out of, and within the shop. Extending the overhead crane outside the building permits loading and unloading railcars.

Most railroads want their own fabrication shop for bridge girders in order to have control over supplying girders in cases of emergencies, such as a bridge washout, and elect to utilize this facility to fabricate girders for their regular maintenance programs.

The Bridge Fabrication Shop should be equipped with overhead cranes and/or rollers for moving steel sections and completed girders, compressed air for tightening bolts, saws for cutting steel, drills to make holes for connections, and gas and electric welding equipment. A supply of new girders of appropriate lengths for emergencies, and steel sections for building up plate girders and gusset plates is necessary.

7.3.4.1 Fabricating New Girders

The steel to fabricate new bridge girders is usually ordered from the manufacturer in the proper lengths and sizes if the bridge is part of a regular maintenance program.

7.3.4.2 Reconditioning Used Girders

Most railroads recover bridge materials from track abandonments, bridge washouts and replacement of bridge spans due to damage or or to increasing the bridge rating. During emergencies, such as bridge washouts or damage by other causes, railroads often need a replacement span to place the line back in service quickly.

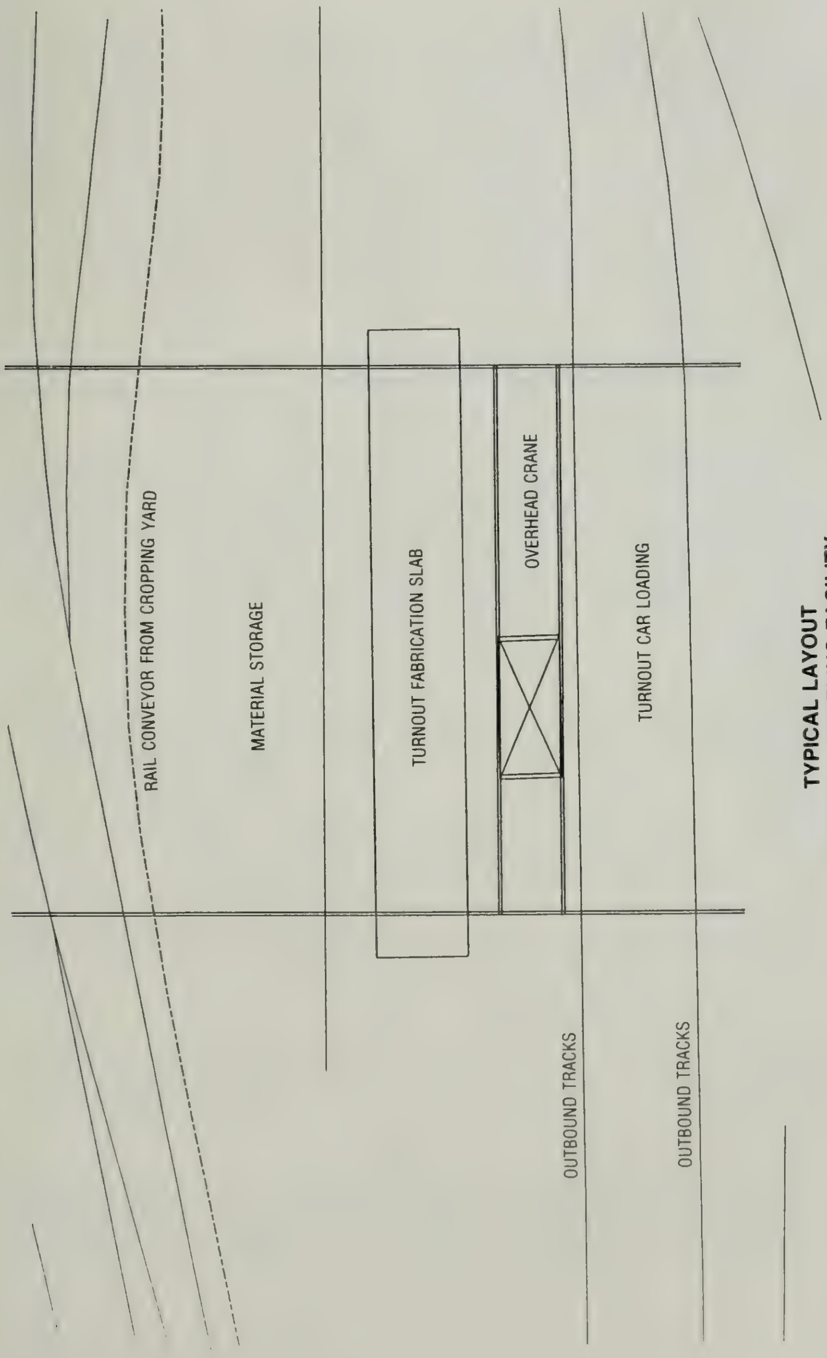
Bridge girder spans recovered from existing track are often repaired and put back into good condition for bridge span replacement, emergencies or regular maintenance programs. The used bridge girder span is straightened, damaged sections are replaced with new steel sections, rivets or bolts are checked and replaced as required and the girder is cleaned and painted. The length of the girder is later adjusted to fit the bridge span where it is to be installed.

7.3.5 Turnout and Track Panel Fabrication Facility

7.3.5.1 General

The purpose of this facility is to build track and turnouts on a large scale assembly line basis more efficiently than can be done in the field. The paneling operation therefore requires a large material supply, with rail and ties normally being received by rail in open top equipment. The OTM may be received by rail or truck. Sufficient ground storage area for track materials must be provided.

Panels fabricated for use in new construction or maintenance programs will normally be loaded as completed and shipped directly to the location where they are to be used. Panels to be used for restoration of track destroyed in derailments or washouts may be shipped to central terminals or held in storage at the fabrication plant until needed. Crane(s) and/or heavy-duty fork lift(s) must be available to load or move the panels to storage.



TYPICAL LAYOUT
TURNOUT PANELING FACILITY
FIGURE 2.

7.3.5.2 Turnout Paneling

Turnout panels are usually built in the open in two or more parts for ease of moving by rail to the point of installation. A typical layout of a Turnout Paneling Facility is shown in Figure 2. The facility has the needed materials for fabrication stored near the assembly area. The switch ties are moved into the assembly area on rollers. The pre-plated switch ties are placed in the proper position using spacers. The frog, switch points, guard rails, stock rails and precut closure rails are moved into place using a crane. The handset spikes are driven with an air driven spike hammer suspended from a frame above the assembly area. The assembled turnout is then loaded onto cars for delivery to the installation site or put into storage for future use. Some railroads use special railcars to transport the turnout panels, and to assist in the removal of the old turnout and installation of the new one. It is desirable to make several turnouts of one size before the tie pattern is modified for another turnout size.

7.3.5.3 Track Paneling

Paneling of 39-ft. sections of track for movement to a construction site for assembly into the track is a fast, and often an economical way, to build yard and other tracks and to restore derailment-damaged track to service quickly. A typical Track Paneling Facility is shown in Figure 3.

Facilities for assembling track panels are usually under cover. Pre-plated ties are moved into the proper position using spacers; the rail is placed on the tie plates; spike holes are pre-drilled; spikes are set by hand and driven with spike hammers suspended from a frame above the assembly area. All ties are usually fully boxed with rail anchors to keep them square with the rails during storage, transportation and placement. Joint bars are usually bolted to one end of each panel, and bolts are loosely fastened in the unused bar holes. Panels are often stored, on the ground or on specially fitted flatcars, with every other panel upside down to make a stable, compact pile.

7.3.6 Site Selection

7.3.6.1 Location

The site for the Plant should be on a main line at a location near the center of the railroad system and have good highway access. It may be desirable to locate the Plant adjacent or in close proximity to a Rail Welding Plant.

7.3.6.2 Size

The size of the property required for the Plant should be consistent with the functions performed and the volumes of materials being handled, and include adequate storage space.

7.3.6.3 Zoning and Permits

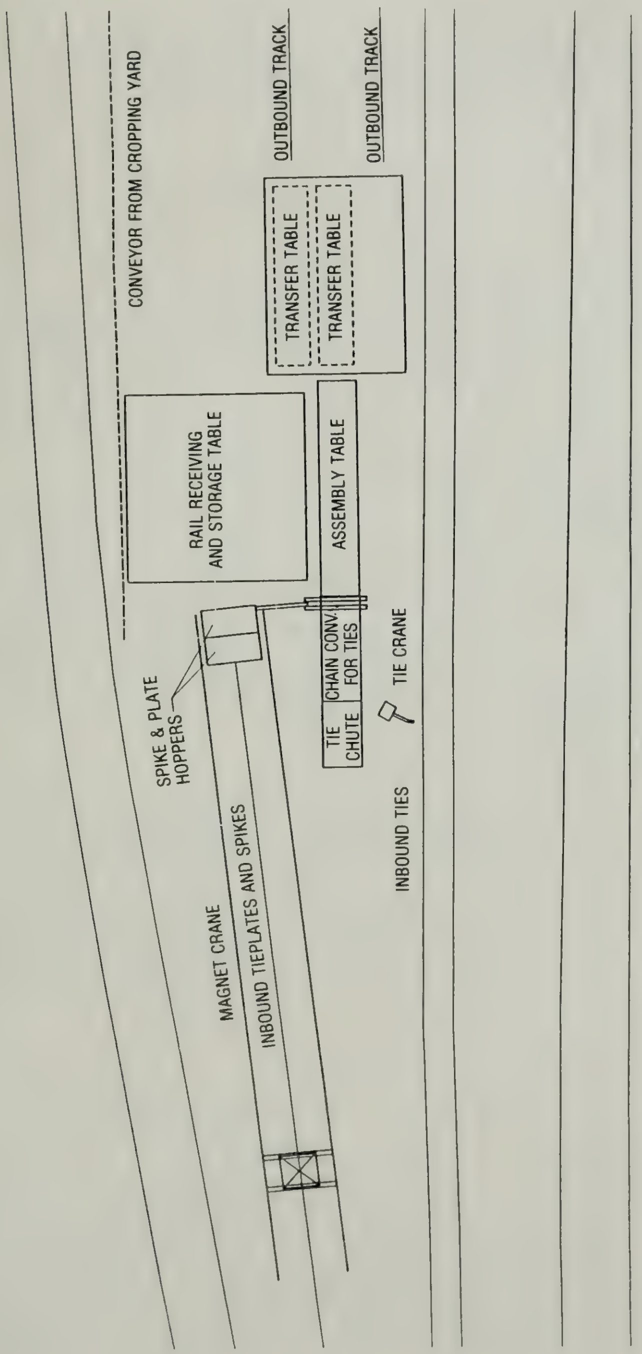
As most governmental jurisdictions have zoning laws it is important to determine if the proposed site is zoned to permit constructing the facility. The facility must also conform to building codes.

7.3.6.4 Environmental Considerations

Chapter 13, Environmental Engineering, AREA Manual for Railway Engineering, discusses environmental considerations in detail. Environmental factors to be considered are air quality, water, noise and lighting, rainfall runoff, archaeological and historical sites, and housing displacement.

7.3.6.5 Utilities

Electrical power compatible with the machinery used is required. Power for inside and outside lighting is also required. Normally the only outside lighting needed is for security. Sanitary sewers, water, and HVAC should be provided in accordance with the facility requirements. Communications facilities within and beyond the operation should be provided for efficiency.



TYPICAL LAYOUT
TRACK PANELING FACILITY
FIGURE 3.

7.3.7 Grading, Drainage, Site Preparation

It is imperative to follow good engineering practices in all elements of design and construction, including preparing and compacting the subgrade, increasing the bearing capacity of soils as necessary, protecting against erosion, proper sloping of cuts and fills, and providing surface and/or subsurface drainage systems.

Disposing of surface and subsurface water is an essential element of design and construction. The subsurface investigation program includes obtaining information on subsurface water conditions and a reliable determination of the ground water table. Historical data that may be available can be very useful to the designer.

Local, state or national agencies may have drainage design requirements and may specify certain design procedures. In the absence of any jurisdictional agency, county or state highway department procedures are suggested for use.

U.S. Geological Survey topographic maps and aerial photographs are useful for analyzing both topography and drainage. They show elevations, contours, bodies of water, streams and other features from which the rate of run-off can be established.

It is desirable that roads and storage areas be surface treated to prevent dust and to expedite movement of material within the Plant.

7.3.8 Track Design

The track layout must be developed to fit the size and shape of the property available for the Plant. The track gradient should be as flat as possible, preferably not exceeding 0.1% grade. The curvature should not exceed 12 degrees. Turnouts should be No. 8, or the size that is standard for the railroad. The mainline should be protected by derail and any track on which people are working should be protected by a blue flag system. A bumping post should be considered for the end of each track. Road crossings should be provided as necessary to facilitate movement of materials.

7.3.9 Storage Facilities

The efficient operation of a Roadway Materials Reclamation Plant is very dependent on proper storage and good housekeeping. Materials held at the Plant for later shipment can be stored either inside or outside. Large items, such as frogs, glued joints, track panels, and other items that will not deteriorate under normal weather conditions can be stored outside. Other items, such as signal equipment, require inside storage. All materials stored inside should be put in a bin, on a shelf, on pallets or neatly stacked in an area convenient for loading out. Material stored outside should be neatly placed in its assigned area. Racks, pallets and paved areas should be utilized to keep the material off the ground and convenient for picking up and loading. All bins, shelves or stacked materials should be clearly labeled with a sign describing the items stored therein. It is imperative that all items be kept in their proper places.

7.3.10 Structures

An office with the normal amenities will be required for supervisory and clerical staff. The size of the office will be determined by the number of employees. Standard office design criteria should be used, including provisions for employee parking. An employee lunch and locker room should be provided at or near the office.

In general, all shop functions should be carried out in a building. The shop buildings can be pre-engineered buildings, modified as required to accommodate cranes and other special appurtenances. Windows and skylights improve the building lighting and contribute to a pleasant environment. The floors should be concrete. All shops, when required, should be served with tracks and hard-surfaced driveways for the efficient handling of materials by rail or truck. It is often possible to pave the track area adjacent to a platform so that it can be used to load either railcars or trucks.

Proposed 1990 Manual Revisions To Chapter 15 — Steel Structures

A new section on Earthquake Design is proposed for Section 1.3 along with the following changes which are proposed to various articles in Parts 1, 3 and 9 of Chapter 15.

Article 1.11.4 (a) (Page 15-1-34)

Reword as follows:

1.11.4 (a) Cross frames and diaphragms, and their connections, shall be adequate to resist forces induced by out of plane bending and the lateral distribution of loads. Connection plates for cross frames and diaphragms between beams or girders subject to out-of-plane bending shall be adequately fastened to the web and both the top and bottom flanges of the beams or girders.

Article 9.1.11.4 (Page 15-9-23)

Insert after the first paragraph of 9.1.11.4:

Out of plane bending may result from restraint provided by crossframes or diaphragms where there is differential deflection of adjacent beams or girders. This may be expected to occur in spans with curved alignment, skews or multiple tracks and has also been observed in single track spans, without skew, on tangent alignment. Out-of-plane bending may cause high stresses in non-stiffened web gaps, unless rigid type connections are provided to stabilize these gaps.

Article 3.6.1 (b) (Page 15-3-14)

The following two paragraphs are to be added to Article 3.6.1 (b):

The fabricator must make certain that structural members are loaded for shipment in a manner that will ensure that they will not be damaged in shipment. The method of loading must not adversely affect the potential life of the permanent structure. Welding of tie-down attachments to a member is prohibited.

The Engineer may request that loading diagrams be furnished to him for unusual or special members. The Engineer may also request the fabricator to notify the Engineer when any member is ready for shipment so that the method of loading can be observed.

Article 1.3.1 Loads and Forces (Page 15-1-7)

Add to subarticle 1.3.1 (a) the following item number 9.

9. Earthquake Loads

Articles 1.3.14, 1.3.15 and 1.3.16 (Pages 15-1-17 and 15-1-18)

Renumber Articles 1.3.14, 1.3.15 and 1.3.16 to 1.3.15, 1.3.16 and 1.3.17 respectively, and insert the following new Article 1.3.14 on Earthquake Loads with accompanying Figures 1.3.14.1 and 1.3.14.2.

1.3.14 Earthquake Loads

(a) Members and connections subjected to earthquake load shall be designed in accordance with the requirements of the "Seismic Design Guide-lines for Highway Bridges," Publication ATC-6, prepared by the Applied Technology Council, Redwood City, California.

(b) The guidelines are based on acceleration coefficients shown in Figures 1.3.14, importance classifications, and seismic performance categories.

(c) Minimum requirements apply to all parts of the United States. More severe requirements apply to selected areas.

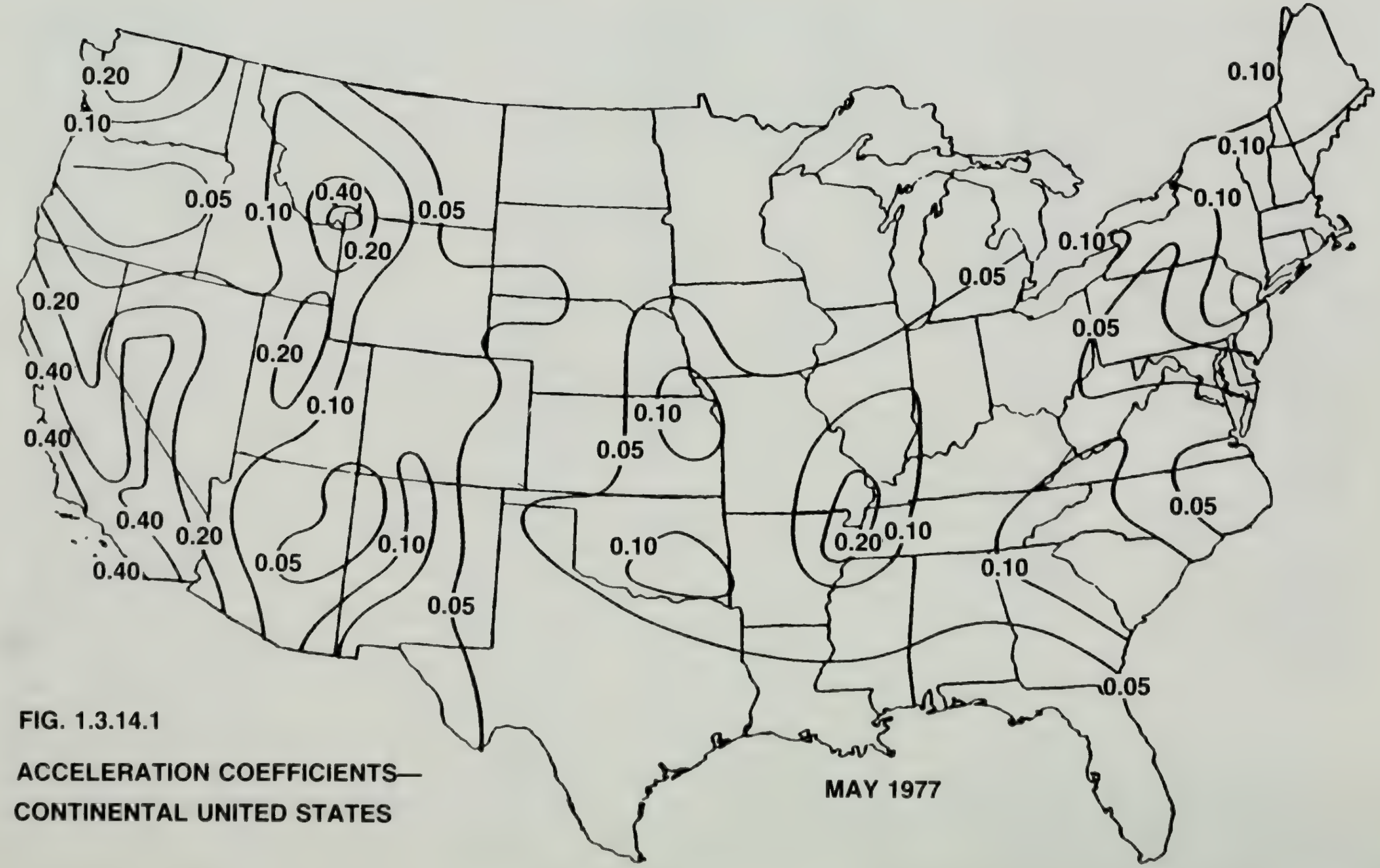


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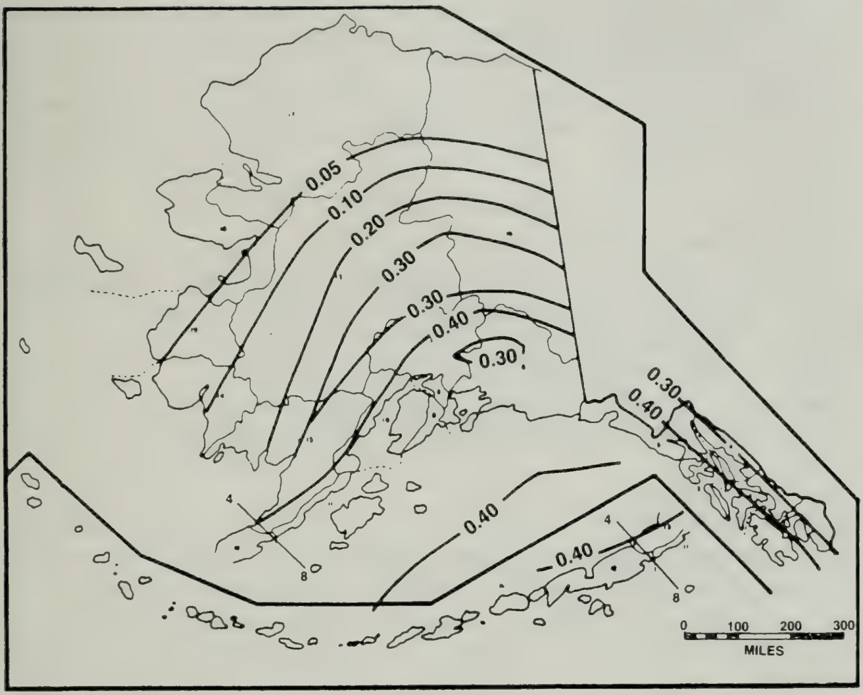
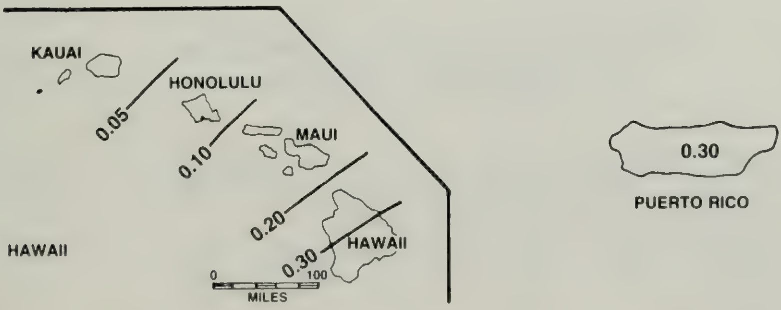
FIG. 1.3.14.1

ACCELERATION COEFFICIENTS—
CONTINENTAL UNITED STATES



ALASKA



HAWAII

PUERTO RICO

FIG. 1.3.14.2
ACCELERATION COEFFICIENTS—
ALASKA, HAWAII, AND PUERTO RICO

(d) The Seismic Design Guidelines are based on Seismic Performance Categories A through D. Category A is for areas of the country having acceleration coefficients equal to or less than 0.09. Other categories are for greater acceleration coefficients.

(e) Methods of analysis are simple for Category A where the design load is small. Increasingly complex analysis methods are required for greater design loads as the categories go from A to D.

Article 1.3.14.3 (existing) which becomes 1.3.15.3 (Page 15-1-18)

Add the following item (e):

(e) Members which are subject to stresses resulting from earthquake load may be proportioned for stresses 33 percent greater than those permitted by (a), but the section of the member shall not be less than that required to meet the provisions of (a), (b), or (c) above.

Articles 9.1.3.14, 9.1.3.15 and 9.1.3.16 (Pages 15-9-13 and 15-9-14)

Renumber Articles 9.1.3.14, 9.1.3.15 and 9.1.3.16 to 9.1.3.15, 9.1.3.16 and 9.1.3.17 respectively, and insert the following new Article 9.1.3.14 on Earthquake Loads.

9.1.3.14 Earthquake Loads

The "Seismic Design Guidelines for Highway Bridges." Publication ATC-6 has been adopted for use since it gives a workable system for seismic design for normal bridge types.

The provision of the Guidelines apply to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 500 feet; other types are discussed in the ATC-6 commentary.

No attempt has been made to include excerpts from ATC-6 in Chapter 15 because of the inherent danger in citing specifics out of context. The user should be familiar with the entire content and the limitation of ATC-6 before using any part of it.

Proposed 1990 Manual Revisions To Chapter 28—Clearances

The dimensions of the recommended base car as outlined in the diagram of Section 3.3 "Suggested Method for Charting All Obstructions" (Page 28-3-10), and upon which *Railway Line Clearances* published clearances are based is proposed to be revised to:

"Published Clearance Outline—based on 73′-0″ car with 50′-0″ truck centers."

NOTES

NOTES

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AMERICAN RAILWAY
ENGINEERING ASSOCIATION

BULLETIN
No. 726

VOL. 91 (1990)
MAY 1990

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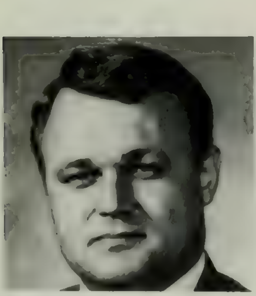

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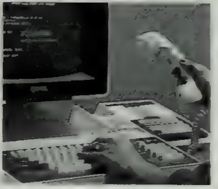

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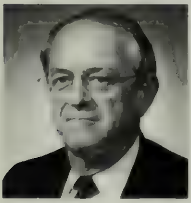
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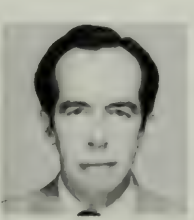
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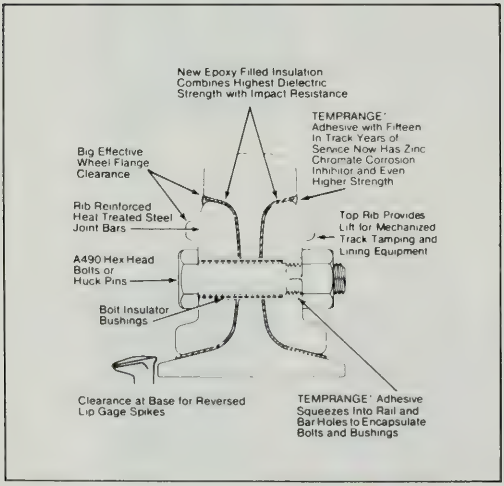

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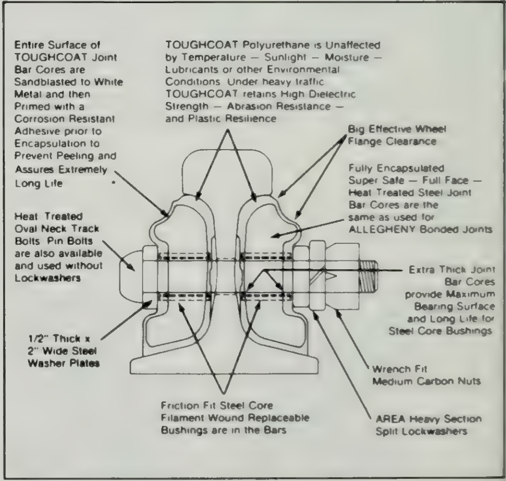

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Cover Photo: Only 42 miles by rail from the Guatemalan border, National Railways of Mexico train No. 102 heads east over recently relaid welded 100 lb. rail at Huehuetan. The Sierra Madre mountains form a back drop for this tropical scene on the Pacific coastal plains. See page 129 for more information.

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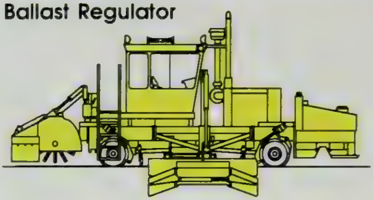
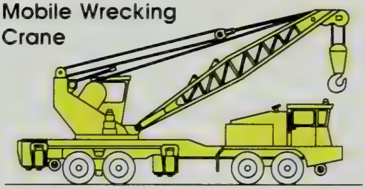


Mobile Wrecking Crane



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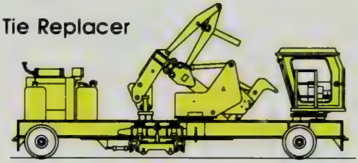

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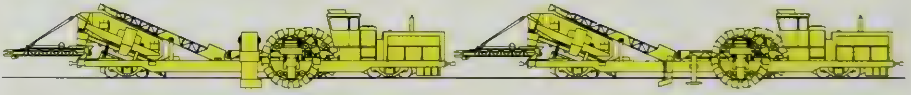

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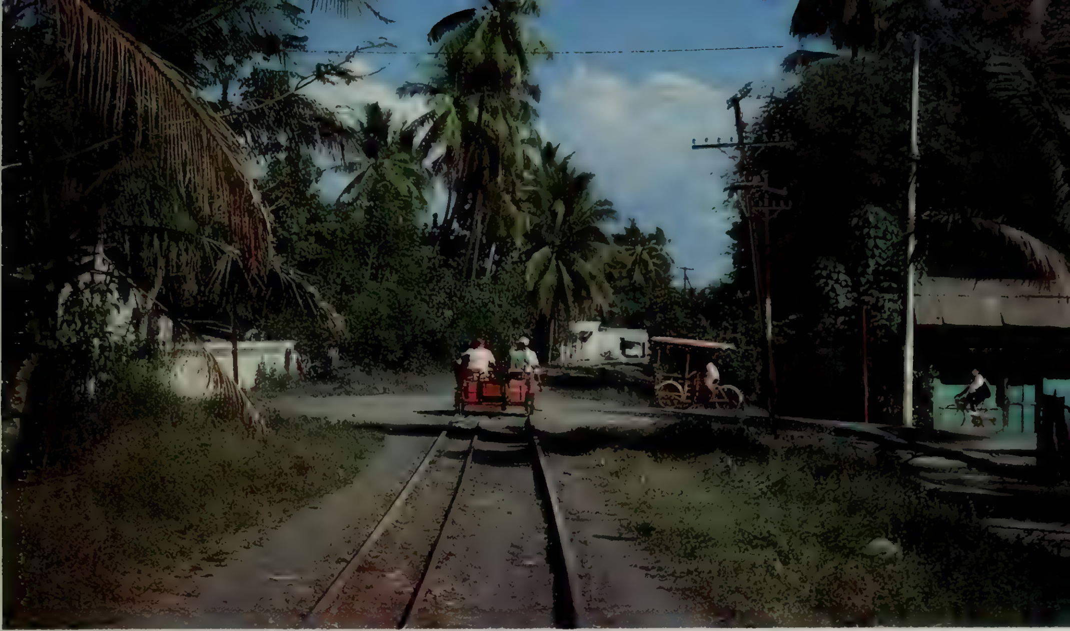
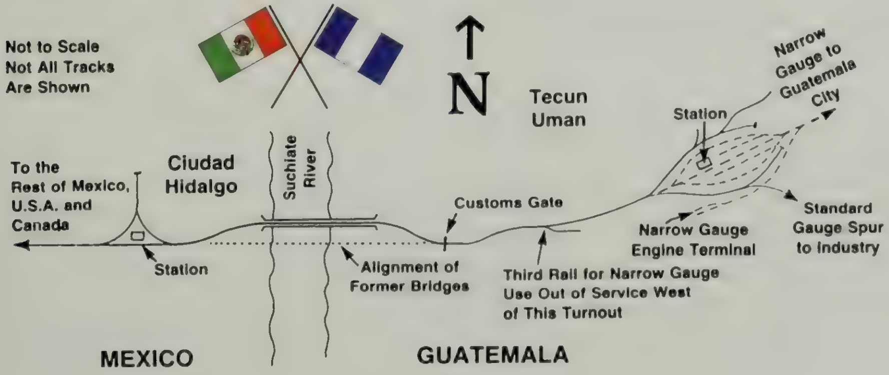


Photo 1. Motor car heads east through residential area of Tecun Uman, Guatemala just east of customs house gate.

To the Southern Extreme of Our Continuous Standard Gauge Network The Mexico-Guatemala Border

There is often a fascination with looking at a trail, or road, or railroad track that is unfamiliar to us and wondering, where does it go, as its image vanishes in the distance or around a curve. We that work on the continuous connected structure that is the North American standard gauge railway network can sense this fascination, for no one individual is familiar with all of it. There is always another line that we have seen at a junction point or crossing but have never travelled. We know that the tracks do not go on forever, and there is a curiosity to know where they finally end up. The extreme southern end of our continuous standard gauge network occurs near the Mexico-Guatemala border a few miles inland from the Pacific, and this photo article has been put together to give all AREA members a little familiarity with this location.

The North American standard gauge network ends at a railway yard about a mile beyond the Mexican Border in the city of Tecun Uman, Guatemala, which is on the East bank of the Suchiate (soo-chee-áh-tay) River across from Ciudad Hidalgo, Mexico. (see map below) At this yard traffic can be transloaded into cars of the 3-foot gauge Guatemalan railways.



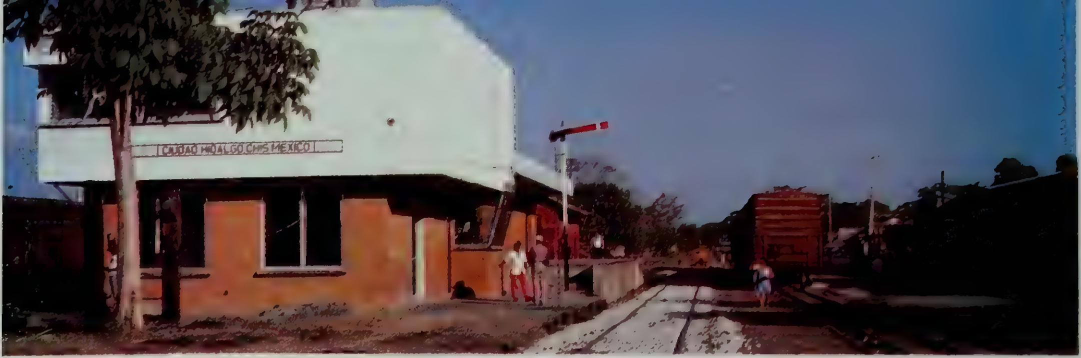


Photo 2. Station at Ciudad Hidalgo, Mexico, looking east down mainline towards Guatemala.

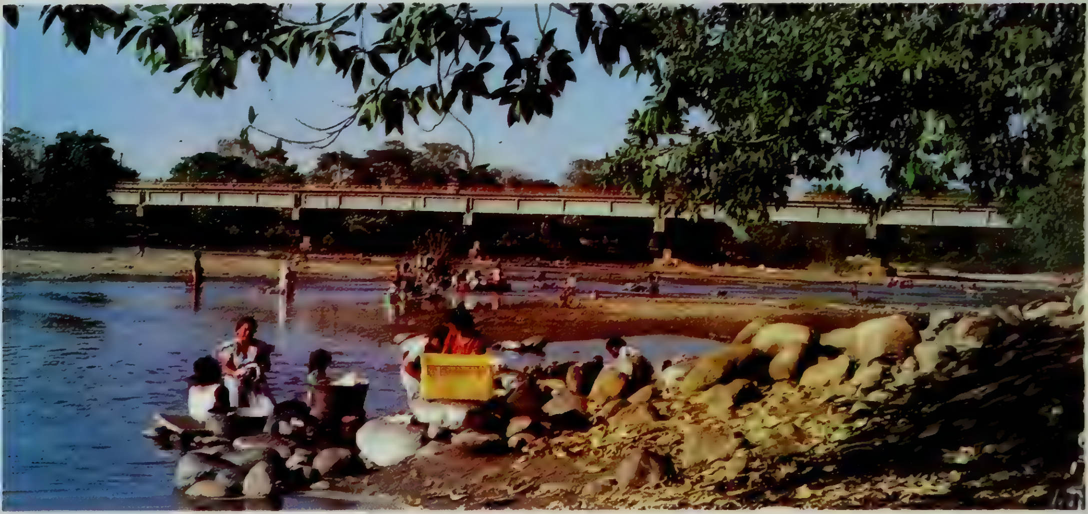
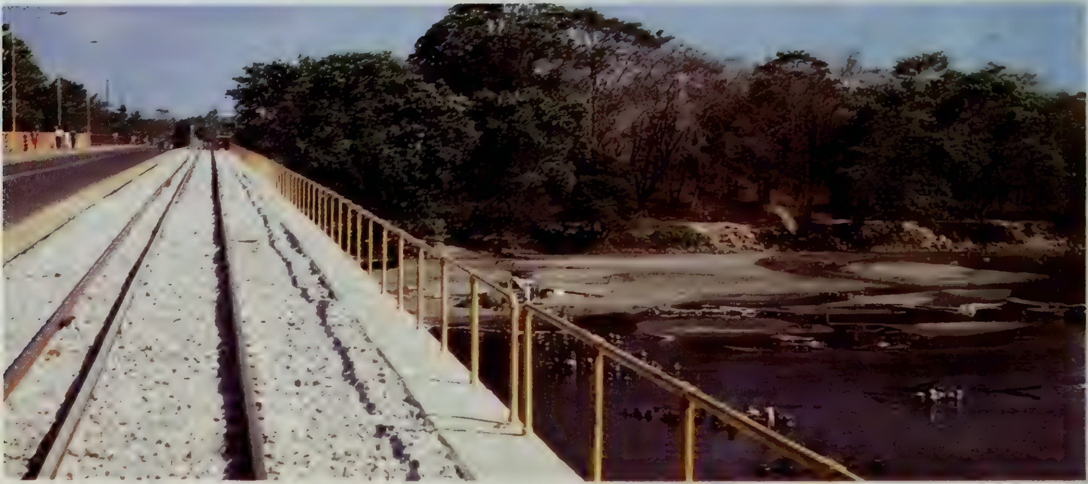


Photo 3. The Suchiate River Bridge.

Above: Women wash clothes and children swim in the clear, shallow mountain-fed waters of the river.

Photo 4. Below: On the south side of the existing bridge are evidences of previous timber pile trestles across the river and partially-buried steel girder spans from past floods which took out the bridge several times. It is said that when the bridge was timber it got to be a contest to beat previous records for reinstalling the timber bridge after floods. The record was ten days.



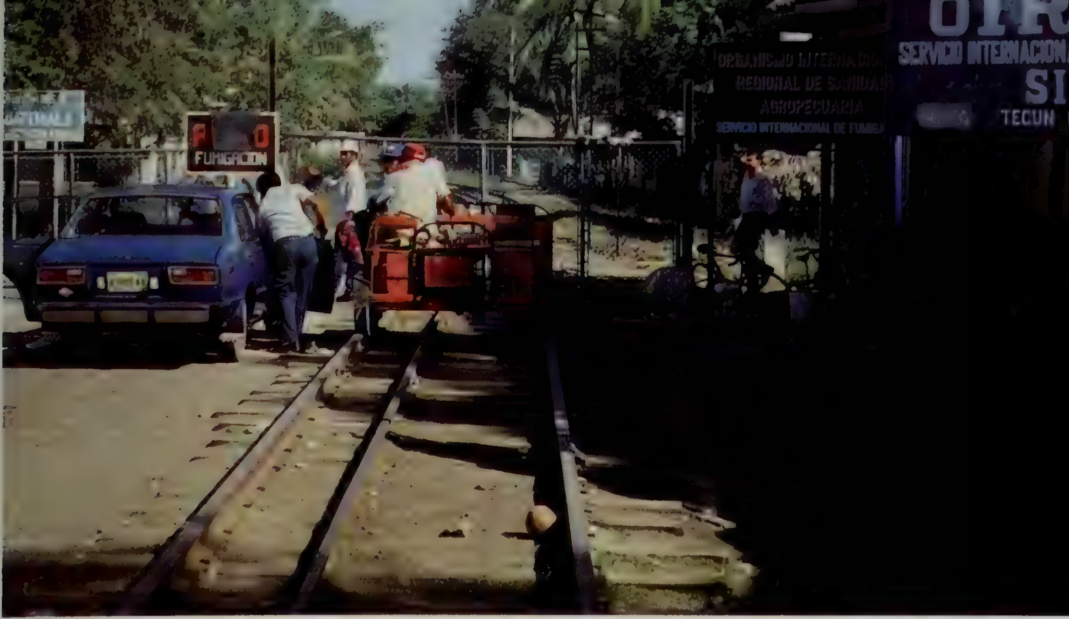

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Photo 5. Border customs house on Guatemala side of Suchiate River.

A trip by rail across the Border from Mexico into Guatemala begins at the Ciudad Hidalgo station (see photo 2) where there is a "Y" track to the north. The rail is approximately 85 lb., except that the third rail for three foot gauge, which is not in use here, is a much lighter section. Starting east towards Guatemala, the eastern leg of the "Y" comes onto the main line, which continues over a road crossing and through a small "S" curve leading up to the Suchiate River bridge.

The Suchiate bridge is a combined road and highway concrete structure, two highway lanes on the north and the railroad track on the south. The railroad side is ballasted deck (see photos 3 and 4). The west end of the bridge is near the western bank of the river, but the east end of the bridge goes over a wide flood plain which unfortunately had a growing population development below.

Photo 6. Standard gauge tracks go on each side of narrow gauge yard at Tecun Uman. Looking east from west end of yard.

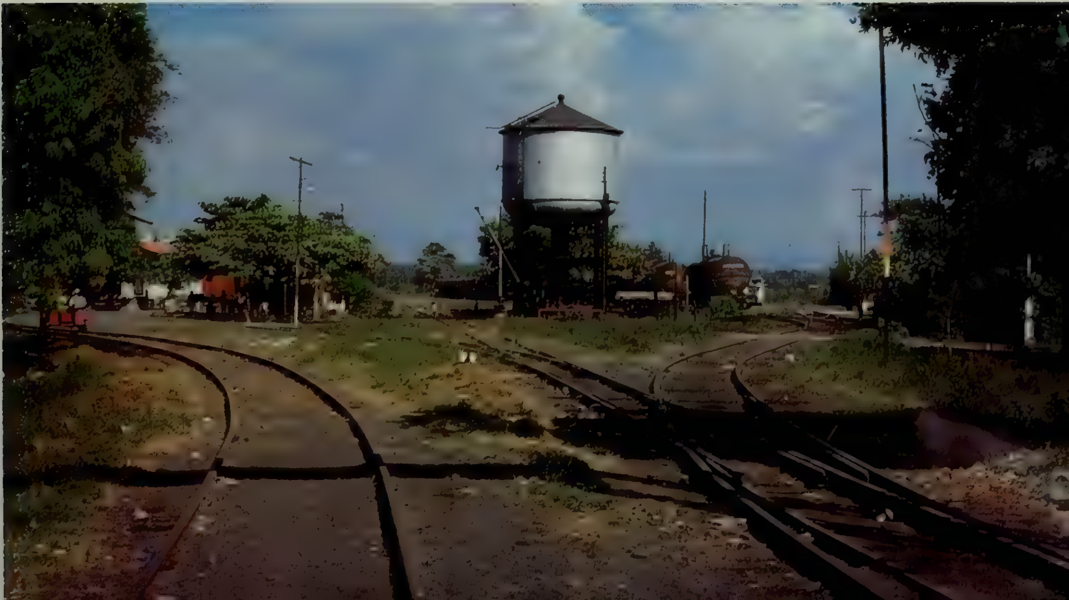


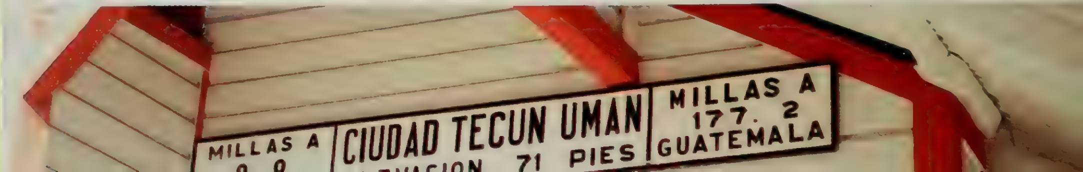
Photo. 7. Standard gauge motor car and narrow gauge box cars on north side of yard at Tecun Uman.

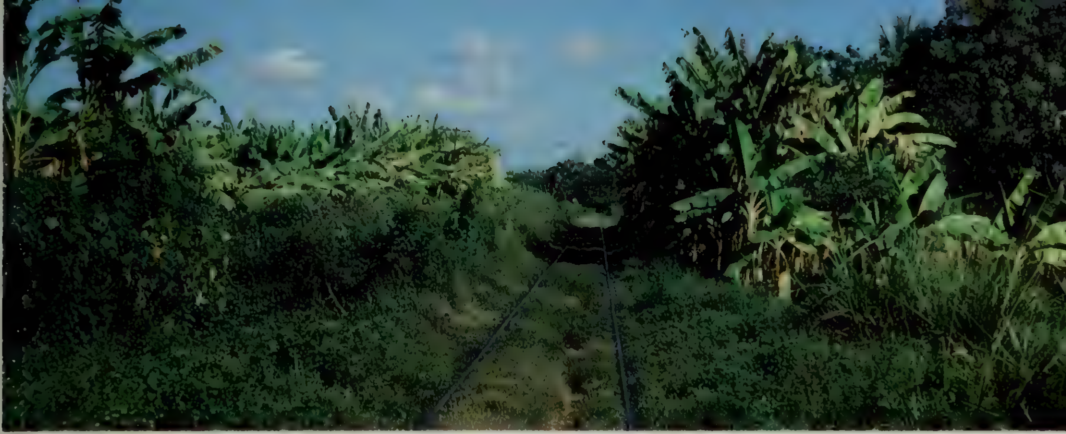
There is a short "S" curve to the right after the Guatemalan end of the bridge leading to the customs post, with a gate across the track (see photo 5). After the gate is opened there is another "S" curve, this time to the left on a narrow right of way through a residential area of Tecum Uman (see photo 1). This is followed by a long curve to the left after a turnout to the right. The third rail for 3-foot gauge equipment was not complete through the turnout, but from this point on east, 3-foot gauge equipment could operate.

At the west end of the yard there are turnouts which put standard gauge tracks on each side of the narrow gauge facilities at Tecun Uman (see photo 6). The yard has cream-with-orange trim station and other buildings with shade trees to provide relief from the tropical sun (see photos 7 and 8). On the north side of the yard there are two industries served by standard gauge track. The standard gauge track on the south side of the yard has a turnout leading to a long standard gauge spur ending out of sight across fields to the southeast, apparently at some type of factory. The yard had interchange tracks with long lines of tank cars and also some tracks with containers. We were told that most of the interchange here was with Guatemalan trucks, although there is some with the railway. Towards the east end of the yard the narrow 3 ft. gauge tracks gather themselves together and head off into Guatemala through the coastal plains. Near the station there is a small servicing facility to the south for narrow gauge engines where two tracks go over a servicing pit.

This yard, the southern end of our continuous standard gauge network, is connected to Mexico City, Chicago, Montreal, and every other rail point on the network, and is the reason the AREA symbol includes up to the Mexico-Guatemala Border. The Mexican line that heads to this area from the rest of the North American network is the subject of the article on the next page.

Photo 8. Station sign at Tecun Uman, Guatemala





Scene at Km 402, 2 Km ahead of gang

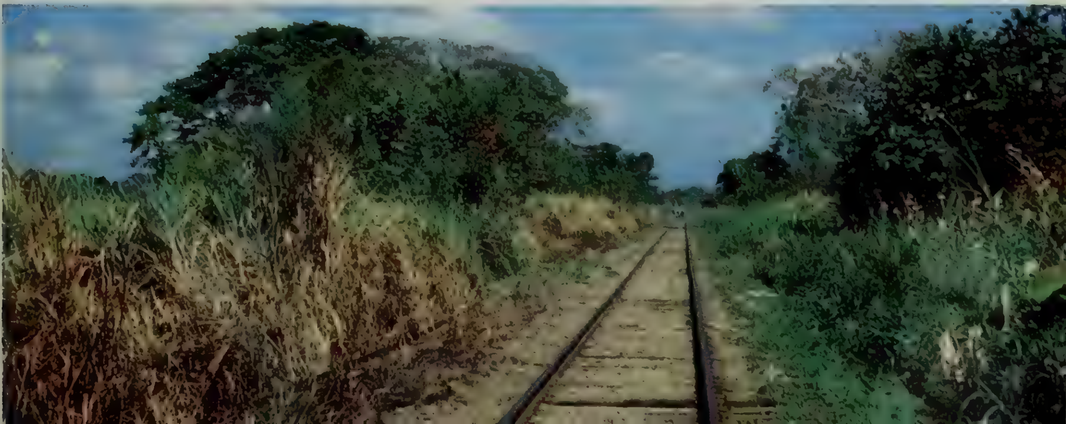
National Railways of Mexico Upgrades Line to Guatemalan Border

The National Railways of Mexico is in the process of upgrading with heavier rail its line to the Guatemala border, which carries two passenger trains in each direction per day in addition to freight traffic. The line runs 285 miles from Guatemala to a junction near Ixtepec with the line across the Isthmus of Tehuantepec (the narrowest part of Mexico) between the Pacific and Gulf. Most of the line runs parallel to the Pacific coast, just south of the base of the towering Sierra Madre mountain range. The runoff from storms in these mountains presents hydrological challenges at some stream crossings.

The picture above shows the condition less than two miles in front of the rail-laying gang and the picture below shows the condition less than two miles behind the gang. In this rehabilitation, jointed 85 lb. rail is being replaced with used 100 lb. rail rolled in 1962, much of it in welded strings. Derailment experience has been much reduced since the rail relay has taken place, and the ride is much smoother. The photo on the cover of this bulletin shows a train passing over a section of the line recently rehabilitated with welded rail. A small surfacing operation is taking place to level out low spots remaining from the old rail. Vegetation control in the tropics is a continuing battle due to moisture and the all-year-round growing season, and should not be judged by the standards of temperate areas.

This rehabilitation project should assure a long period of good quality track on this southernmost line on the railway network of our continent.

Scene at Km 398, 2 Km behind gang



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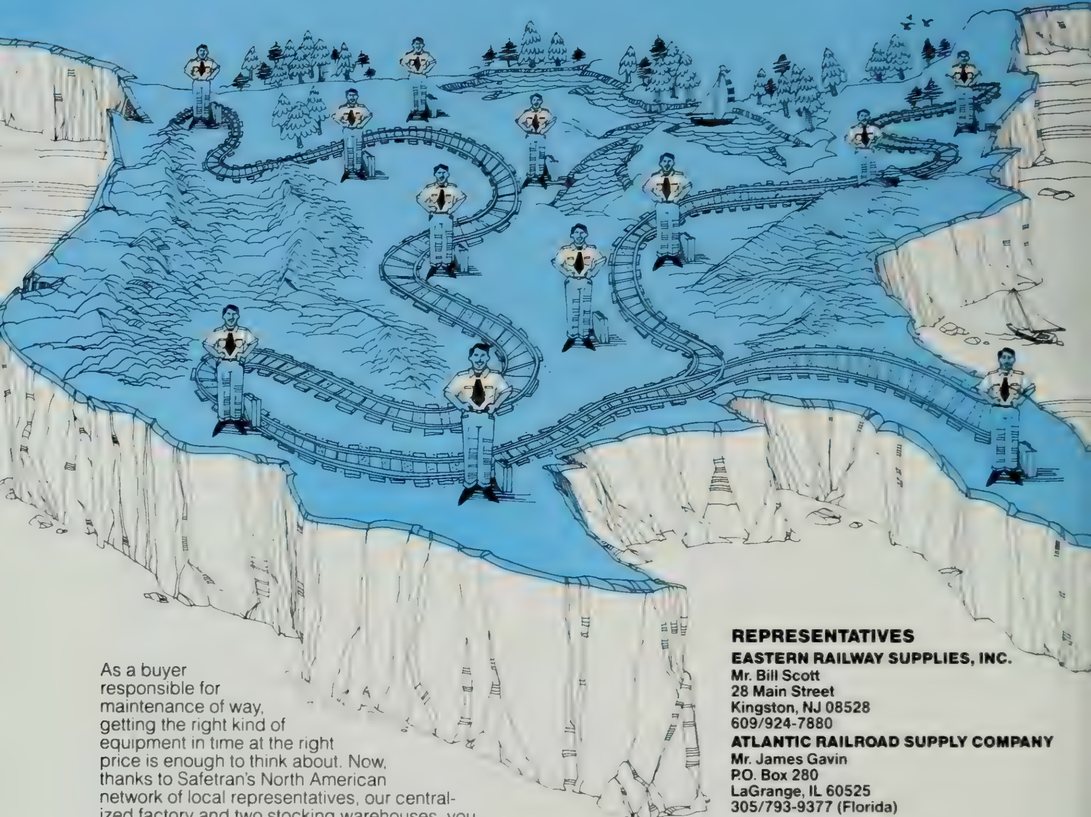

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Thoughts at 186 m.p.h.

Travelling towards Paris at 186 mph at sunrise on a hazy morning in the cab of a TGV Atlantic train. Most of the thoughts are the same as at 160 m.p.h. (see page 330 Bulletin 717, October 1988), and the ride is still as smooth, if not smoother. Handwriting samples below show how smooth it is. Again there is the realization of the inherent potential of the rail mode, how the basic track structure of rails, ties, and ballast, maintained by well-proven equipment used extensively on regular-speed railway lines, can easily handle these speeds. And not only these speeds, but far higher ones.

On December 5, 1989, this same type of equipment achieved a speed of 300 m.p.h. (482.4 k.p.h.) on this identical type of track structure. And even this speed did not over stress the track. After the 300 m.p.h. test, TGV Design Engineer Francois Lacote said, "The great impression one is left with is that there is no impression. You see the countryside going past faster, but you get used to the speed very quickly. There is a margin available at 300 m.p.h. and there is a long, long way to go before the limits of wheel-rail technology are reached." (The article on pg. 174 covers this high speed run in detail.)

The TGV is now the fastest guided ground transportation vehicle ever to have carried people (it carried over 50 on its record run), and is considerably faster than any mag-lev vehicle except small, unmanned test vehicles.

An important advantage of wheel-rail is its ability to continue onto regular lines without stopping. In France this involves 137 m.p.h. running over regular lines also used by freight trains, as well as local passenger trains. The four pictures on the next two pages show one location north of the Le Mans station in a single 45-minute period, and show the multiple uses of which railways are capable. France has demonstrated that a double-track line built in the 19th century can safely and simultaneously serve as a facility for the movement of 137 m.p.h. express trains, heavy freight trains, and local trains stopping at numerous intermediate stations. Grade crossings have been eliminated where speeds are 124 m.p.h. or over.

This was written while the train was standing.

This was written while going over 180 m.p.h.

The four photographs on these two pages were taken within 40 minutes of each other on November 11, 1989 just North of the Le Mans station, and show the multiple uses of which railway technology is so capable. Between this point and the present end of the TGV Atlantic line from Paris, TGV trains

running at 137 m.p.h. are intermixed with freight and other passenger trains, including locals, on a single double track line. Local station platforms are passed without reducing speed. At the junction with the new TGV Atlantic line, trains take the diverging side of a No. 65 turnout without reducing their speed.

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BURLINGTON NORTHERN'S ASSESSMENT OF THE ECONOMICS OF HIGH CAPACITY/ HEAVY AXLE LOAD CARS

By: R. R. Newman*, A. M. Zarembski**, and R. R. Resor**

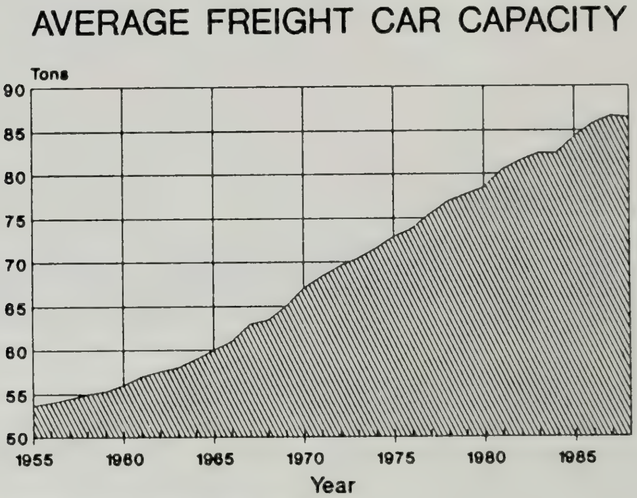
Introduction and Background

North American freight cars and trains have been growing heavier for many years. For the most part this has been a response to competitive pressures, the inflexible nature of train crew costs, and the changing mix of traffic. The shift in traffic towards bulk commodities and unit trains (grain, coal, ore, and aggregates) was one manifestation of this change in traffic mix and its consequent movement toward heavier cars. Figure 1 shows graphically the near doubling of average car capacity since 1929.

This trend to heavier cars has been accompanied by considerable research into the costs and benefits of larger cars and heavier trains. In particular, the issue of increasing car size, and consequently increasing axle loads, has been the subject of much examination and discussion. There s also the related issue of increasing the loading of existing cars.

This issue was raised in 1986 at the Third International Heavy Haul Railways Conference (1) attended by representatives of Burlington Northern's Research & Development Department. In the opening paper of the conference, representatives of Mt. Newman Mining Co. and BHP Melbourne Research Laboratories stated that heavier axle loads were not only **technically feasible but were also economically feasible** under the conditions as experienced, tested, and applied in Australian heavy haul operations.

This paper emphasized that since existing ore cars were only loaded to about 75% capacity, axle loads could be further increased. Following the First International Heavy Haul Railways Conference in 1978, axle loads were increased by the Australians to 33 tonnes (36.3 tons) and this axle load was adopted as a system wide standard. The successful gain led to adoption of an additional axle load increase to 35 tonnes (38.5 tons). Even higher axle loads appeared economically justifiable; however, the 35 tonnes provided an additional margin of comfort below the 37 tonnes (40.7 tons) level at which



Source: Association of American Railroads
Railroad Facts

Figure 1

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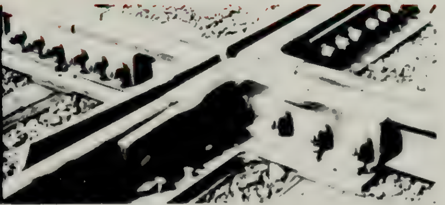
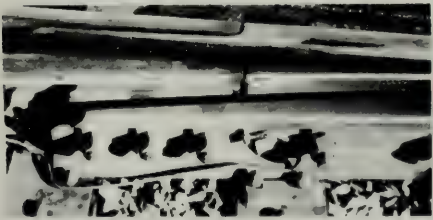


Crossings



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the Australian road's studies indicated that major, cost-impacting bridge upgradings would be required. As illustrated in Figure 2, the main point of the Australian findings was that axle loads between 33 and 37 tonnes (36 and 41 tons) were expected to reduce total railway maintenance and replacement costs. Costs were reduced 1-4% by increasing axle loads from 36 to 40 tons but began increasing beyond that point.

In many ways, BN's Northern Coal Route is similar to the Mt. Newman Railway. It is composed almost entirely of welded rail, the bulk of it being 132 lb. section. Locomotive-borne lubrication is used and profile grinding of the rail is carried out twice per year. The vast majority of the traffic on the line is unit coal traffic. However, since BN's Northern Coal route is not an isolated captive railway, maintenance standards for both track and structures may be less rigorous. Mixed traffic and bridges of varying age and capacity present problems not encountered by the Australian railways.

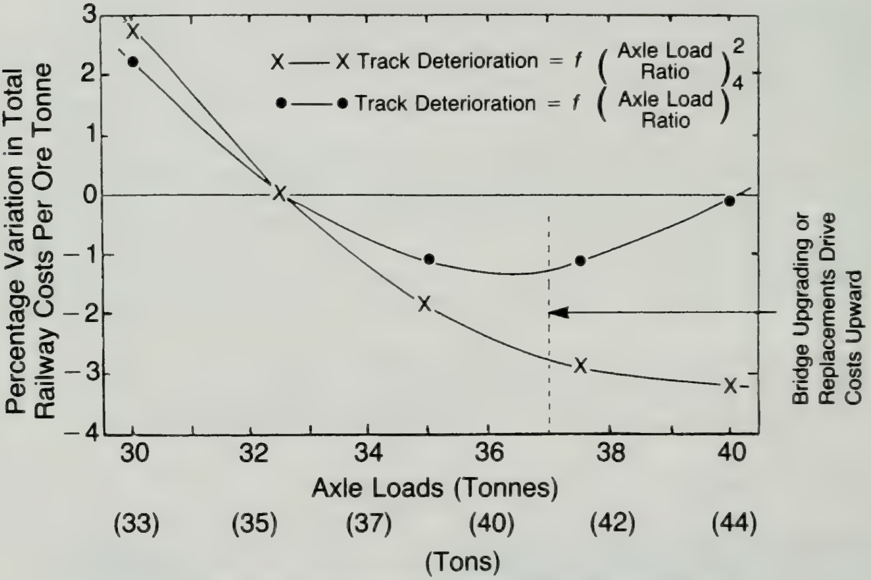

Figure 2. Influence of Axle Load on Total Railway Costs

Heavy Axle Load Coal Study

Recognizing the sensitive issues being raised by the conference paper and historical questions raised by the industry, BN's management decided to formally investigate the possible economic opportunities offered by a policy of increased axle loadings. These economies could be especially significant to BN since it is the largest coal-hauling railroad in the US. A formal, proprietary study of the economic benefits and drawbacks of heavier axle loads was commissioned at BN for bulk commodity coal in the Spring of 1987. **The objective of the coal study was to determine whether cost savings could be achieved by increasing loads in "existing" equipment and by increasing train size for a specific movement over BN's Northern Coal Route.**

To effectively carry out this study an inter-departmental Heavy Axle Load Coal Task Force was assembled at BN with representatives of various departments: Coal Marketing, Costs and Financial Analysis, Transportation, Engineering-Bridges and Structures, Engineering-Maintenance, Operations

Research, Mechanical, Research & Development, and Region and Division Operations. BN's Research and Development Department was charged with managing the study and contracted with ZETA-TECH Associates, Inc. of Cherry Hill, New Jersey to provide detailed engineering and economic analyses. ZETA-TECH was selected because of its expertise in rail and track structure technology and experience with industry engineering and economic analysis models.

Heavy Axle Load Intermodal Study

In September of 1987, following the bulk commodity study, BN's Research & Development Department was additionally requested by Intermodal Marketing to carry out a similar study to assess the implications of "Heavy Axle Load Intermodal Equipment" (3). While 100 ton "twinstack" container carrying equipment had already proven successful in service, its articulated design was known to allow for very high axle loads. The continuing increase in container sizes and payload weights raised significant questions about profitability. **BN's Intermodal Marketing department requested that BN's primary Seattle-Chicago intermodal container lane be selected for the study of the costs and benefits of four different types of intermodal carrying equipment.** (See Figure 3).

- Conventional TTX flat cars (baseline)
- Bulkhead "Twinstack" cars
- High Capacity Well-Type Twinstack Car with 125-ton Trucks
- "Stand-alone" non-articulated double-stack

Fixed tonnage and fixed train lengths were both studied to determine the costs and benefits of operating longer and heavier trains.

ZETA-TECH Associates, Inc. was again selected by BN's R&D Department as the contractor to perform technical assessments and a similar but separate Task Force was assembled to carry out the study. The Intermodal Marketing Department supplied key personnel to the Task Force. In addition to System staff officers, Seattle International Gateway (SIG) and Chicago (Cicero) Terminal were contributors to the study. Trailer train, Transit America, and Gunderson, Inc. provided technical support.

Methodology

From an engineering standpoint, there is no question that heavy axle loads shorten track component lives, increase the rate of degradation of the track structure, and increase the cost of derailments. Likewise, equipment maintenance costs per car will increase with increased car loadings. However, the operating savings that can be achieved by operating fewer but larger cars offer several benefits, including:

—improving the net to tare ratio
—reducing fuel consumption per net ton of coal
—allowing heavier trains to operate without the need to lengthen sidings
—reducing car and locomotive miles operated
—allowing a given quantity of freight to be moved in fewer trains

The key question in evaluating the overall costs and benefits of heavy axle loading equipment is whether these operating savings do, in fact, off-set the increased track and equipment costs from increasing axle loads.

The answer to this question is both service-specific and route-specific. That is because many of the key variables that can affect the outcome of such an analysis are both service and route specific. Therefore, any such detailed study must confine itself to particular routes and services with particular commodity characteristics. Thus, train weight and length are determined by grades and siding lengths (as applicable), maximum feasible wheel loads by the load limit on bridges (which likewise enters into

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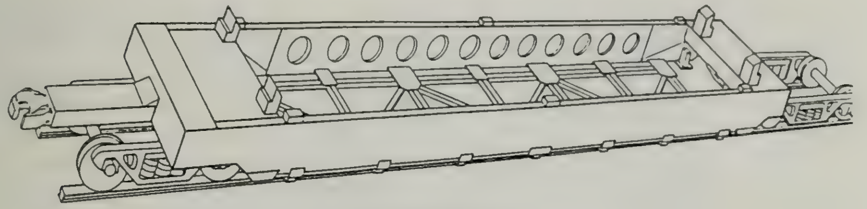
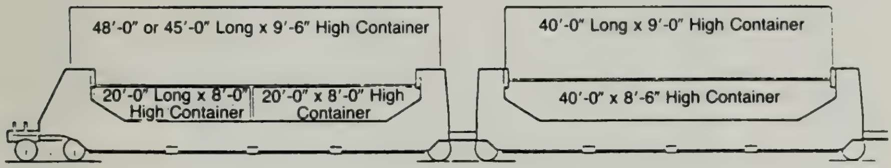
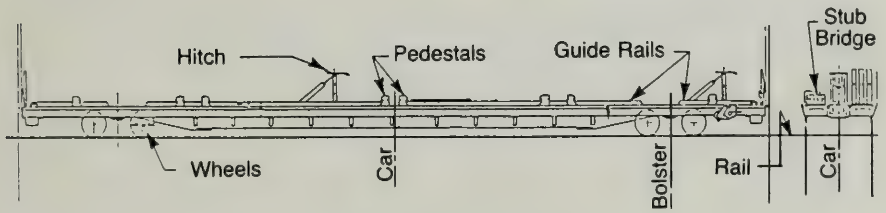


125 Ton IBC Car


48'-0" or 45'-0" Long x 9'-6" High Container
20'-0" Long x 8'-0" High Container
20'-0" x 8'-0" High Container
40'-0" Long x 9'-0" High Container
40'-0" x 8'-6" High Container

100 Ton Bulkhead Car



89 Foot TTX Car

Intermodal Car Types Evaluated
Figure 3

the economic analysis). Commodity characteristics detemine product density, and thus car capacity and weight, as well as loading/unloading restraints. Track maintenance costs are determined by the weight and type of rail, the age and condition of the ties, ballast quality, and the quantity of lubrication on curves. Conclusions about the optimum train weight, car weight, and train length for one route, commodity, and service, are not necessarily applicable to another.

This paper presents the results of two economic studies carried out on specific routes on the Burlington Northern Railroad. These studies addressed distinctly different types of service: **bulk commodity-unit coal service and high speed intermodal service.** Specifically these studies were:

1. An **annual movement of a fixed volume of coal** from a mine in the Powder River Basin (Wyoming) to a Power Plant in Minnesota. This movement currently moves in unit trains of 100 ton capacity cars.

2. An **annual movement of a fixed volume of containers** from port (Seattle, WA) to major transfer point (Chicago). These containers currently move in conventional double-stack equipment.

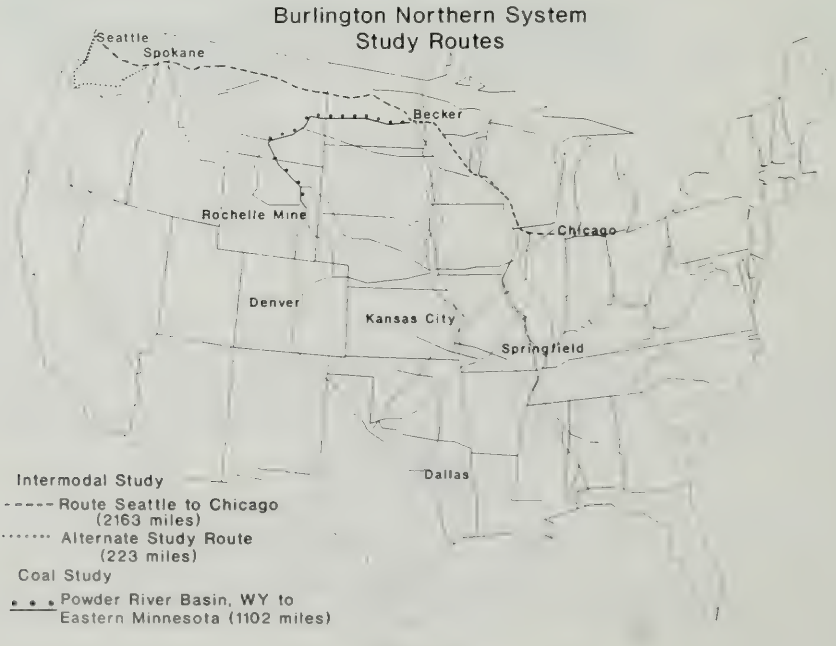
The intermodal analysis was different from the coal study in that coal costs and benefits could be expressed on a per ton basis, whereas **intermodal costs must be expressed on a per container (per box) basis.**

In carrying out this analysis, a methodology was developed to analyze the overall systems costs of a specific move or set of moves, taking into account all of the operating, capital, and maintenance costs of that move. This methodology included a sensitivity to axle load, which allowed for its direct application to the two studies noted above.

The methodology presented in this paper is intended to analyze a specific movement(s) of commodities by taking into account all of the *direct* operating, capital, and maintenance costs of that move(s). Given the overall emphasis of increasing train size and/or increasing car capacity, this methodology has a major focus on the effects of axle load (and specifically increases in axle load) on these costs. Therefore, by carrying out a comparative overall analysis of this movement and examining various combinations of train and car sizes and configurations, it is possible to **determine the optimum axle load** (car net load), train size, and train configuration (to include car type) for specific commodity movement over a particular route.

As noted above, two different sets of moves were analyzed: a contract **coal movement from Rochelle Mine, in Wyoming's Powder River Basin, to a power plant at Becker, Minnesota,** and a movement of **ocean containers from Seattle International Gateway to Cicero yard in Chicago** over Burlington Northern's "high line" route. These routes are shown in Figure 4.

Only the direct costs of movement were considered; corporate overhead and those costs (such as snow removal) that logically do not vary with train size or wheel load were excluded.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Spokane
Becker
Rochelle Mine
Chicago
Denver
Kansas City
Springfield
Dallas
Intermodal Study
----- Route Seattle to Chicago (2163 miles)
······· Alternate Study Route (223 miles)
Coal Study
• • • Powder River Basin, WY to Eastern Minnesota (1102 miles)

Figure 4

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Direct costs of movement considered in the analysis included:

—crew wages
—fuel cost
—equipment maintenance expense (direct cost only, on a per-mile basis)
—equipment ownership expense (weighted average purchase price amortized over the economic life of the equipment)
—track maintenance expense (a composite of expensed items, including maintenance performed by divisional forces and capital expenditures)
—an annualized cost of required bridge upgrading or replacement to carry the heavier loads
—increased operating costs as a result of any bridge slow orders made necessary by the heavier wheel loads
—an estimated annual cost of derailments and delays due to heavier loads, principally from additional wear and tear on the equipment

Use of Economic Models

A key analysis tool used in the study was the **RECAP (Rail Energy Cost Analysis Package) model** developed by the Association of American Railroads (4). RECAP is a complex and sophisticated simulation program which allows detailed development, of the costs of operating a particular consist over a particular route.

The RECAP model [and its operations simulator, the **Train Energy Model (TEM)]** was used to calculate fuel consumption, crew cost, overall running time, and operating speed for a defined consist operating over a particular route. RECAP was also used to estimate track and equipment maintenance costs for existing equipment; however, the model was not able to evaluate the effect on either track or equipment of incremental increases in car weight. It also could not estimate the specific costs of bridge maintenance and rehabilitation, and it could not quantify the effect of derailment cost.

Development of "Damage Factors"

Where RECAP does not possess the sensitivity to produce cost estimates, external engineering analyses were performed. RECAP's track cost estimation procedures [which are based on an earlier track maintenance costing model (TMCOST)] have been the starting point for the development of engineering-based **"damage factors"** that specifically relate wheel loads and track component lives. A similar methodology was used for equipment; the anticipated effect of heavier loading on each car component category (wheels, brakes, bearings, car body, couplers) has also been quantified. Anticipated increases in bridge maintenance and rehabilitation costs were estimated with the assistance of Burlington Northern's structural engineering group. The expected increase in derailment costs was calculated from historical data by applying the damage factors developed in the track and structural analyses. The study made use of specific Burlington Northern actual costs or calculated costs which have been adjusted to reflect BN costs.

With the aid of this and other computer simulation techniques, the operation of a range of car weights, train weights, and motive power assignments were examined on the two study routes. For each simulated consist, cost estimates were made for crew wages, fuel, equipment maintenance, track maintenance, and the ownership cost of the equipment. In addition, a supplemental engineering analysis was carried out for the increase in track, bridge, and equipment maintenance costs to be expected with heavier car loading. Expected increases in derailment costs were also estimated. A special assessment was also made, in the Coal Study, of options for wheel sizes (diameters 36″ to 40″). **The analysis indicated that the retrofit of new trucks and larger wheels could not be justified on economic terms.** Finally, all costs were annualized for ease of comparison.

Analysis Assumptions and Bounds

It is necessary in undertaking any quantitative engineering analysis to document the assumptions made, the factors held constant, and the bounds of the analysis. This study has confined itself to following current BN practice. Crew agreements, equipment maintenance, and track maintenance follow present Burlington Northern practice. Guidelines on maximum train weight and length are those the railroad uses in assigning motive power.

For the two specific studies presented in this paper the following constraints were imposed:

Unit Coal Train Movement

The coal move being studied involved the shipment of more than 3 million tons of coal per year from the Powder River Basin to Minnesota.

Existing equipment was used in the analysis, since the Burlington Northern had a surplus of both hopper cars and high-side gondolas during that period of the study. Two "typical" car classes were used in the analysis: the H450 hopper and the J401 gondola. Both are fitted with rotary couplers and are rotary-dumped, even though the H450 has bottom-dump gates. The cars are identical in length and very close in weight. The principal difference between them was per mile maintenance expense. The H450 was a bit more expensive to operate; presumably this difference results from the presence of the bottom discharge gates.

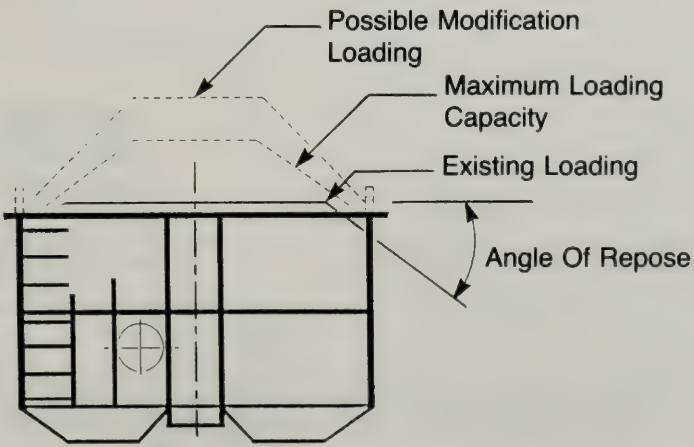
Purchase price data was more readily available on the H450 so it was used in all the analyses.

Since the objective of the study was to determine the costs and benefits of heavier axle load limits in **existing equipment,** a critical concern was volume of coal that could be physically accommodated in the H450 hopper. The manufaccturer was contacted to determine the interior volume, which was found to be 4008 cubic feet. A literature search indicated the angle of repose for coal to lie between 25 and 35 degrees.

The volume of a pyramid with a base the size of the top of the coal car was calculated for any assumed angle of repose by use of the formula:

$$V = ((3L - W) / 12)^2 W \tan(\alpha)$$

where V = volume, W = width of car, L = length, and α is the assumed angle of repose (see Figure 5).



Maximum Feasible Increase in Coal
Figure 5

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Since coal cars are subject to vibration while moving, the lower angle of repose was chosen. The volume of a pyramid with a base the size of the top of the car, and a slope of 25 degrees, was calculated to be 492 cubic feet.

Coal from Rochelle Mine and from other mines in the same area, loaded into hopper cars, has an average density of 50 lbs. per cubic foot, **yielding a maximum feasible load of 112.5 tons of coal in each car.** (In some parts of the Powder River Basin, coal density ranges up to 57 lbs. per cubic foot. With this heavier coal, maximum car load could reach as high as 128.25 tons. See Table 1 for volume and weight calculations.)

Table 1: Calculation of Car Volume and Load

Car cubic capacity: 4008 cubic ft. (mfr. measurement)
Coal angle of repose: 25° - 35°
Volume of coal heap at 25°: 492 cu. ft. (see Figure 5)
Total available volume: 4500 cu. ft.
Powder River coal density: 50 lbs./cu. ft.
Effective maximum load = 4500 × 50 or 112.5 tons

Modifying the car sides was also evaluated during the course of the study but dropped from consideration due to loading tipple height restrictions and related costs for modifications.

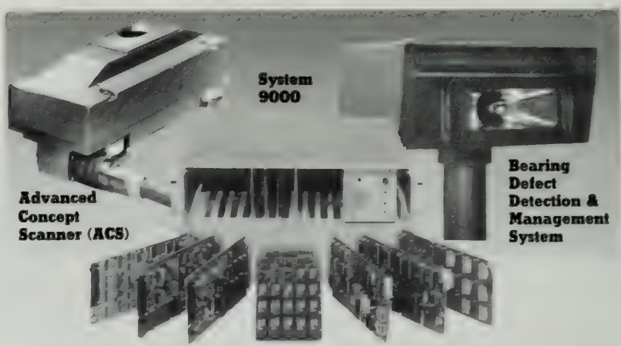
Motive power was assumed to consist entirely of SD 40-2 locomotives. These are in fact the primary coal train power and are 'typical' locomotives in price, performance, and per-mile maintenance cost.

Approximately 300 round trips per year are required for coal moving in BN's existing "100-ton" cars (actually carrying 101.5 tons of coal). By increasing car capacity (through increasing the net load in existing equipment) and/or increasing train weight fewer round trips will be required to move a fixed annual tonnage. Table 2 shows the configurations and gross trailing tonnage of all consists successfully simulated.

Table 2: Train Weights and Annual Trips Required

Case	*Car Weight (Tons)*	*Cars/Train*	*Trailing Weight (Tons)*	*Trips/Year*
A	100	115	15,000	296
B	105	109	14,769	296
C	110	104	14,528	296
D	125	92	14,306	296
E	105	115	15,582	282
F	110	115	16,157	269
G	101.5	115	15,180	292
H	112.5	115	16,444	263
I	101.5	129	17,028	260
L	112.5	120	17,160	252
M	112.5	118	16,874	256
O	125	111	17,260	245

Baseline costs were calculated for current operating practice, a 115-car train of approximately 15,000 gross tons. Maximum train length was set at 120 cars, which required siding lengths of 7300 feet.



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Weight, rather than length, was found to be the limit on train size. Thus, total train weight was defined by the power, train weight, and helper configurations required to maintain the BN's drawbar force limits of a maximum of 375,000 lb. Simulation showed that grade, curvature and rolling and bearing resistance limited train size to approximately 17,100 tons.

A complete set of TEM and RECAP runs were carried out for each of these configurations, and the corresponding fuel, crew, and operating costs, calculated.

Intermodal Container Movement

The Intermodal study examined the movement of a fixed annual volume of containers between Seattle International Gateway and BN's Cicero Yard in Chicago. **In this analysis, four different types of equipment were evaluated and compared;**

- conventional TTX 89′ flatcars
- 100 Ton bulkhead twinstack equipment
- 125 Ton IBC twinstacks
- new design "stand-alone" twinstack car

To perform a comparative analysis of equipment types, it was necessary to define an average trainload of containers based on the operating characteristics of BN's container trains. In this analysis initially all spaces on eastbound trains were assumed to be filled with loaded containers and westbound trains assumed to carry only empty containers. (This assumption was subsequently modified to reflect rapid growth of loaded westbound containers.)

A critical assumption in this analysis was the weight of the containers—and thus the carloads—simulated. Using actual container weight data, average container weights were calculated together with the distribution of the container weight. This distribution, based on actual weighed container samples, follows:

Container Type	Mean	Std. Deviation
20 foot	32,360	13,730
40 foot	34,284	13,076
full sample	33,863	13,245

Noting that both twinstack cars and TTX flats can carry combinations of 20-foot and containers 40-foot, and that, with some exceptions, the loading process was random, a Monte Carlo simulation was used to evaluate the distribution of car loads. Using three pools of containers, the containers were "loaded" onto a car as one of two combinations; two 40-foot containers or two 20-foot containers and a 40-foot container. Varying mixes of containers were set by changing the probability of a 20-foot pick. Figure 6 presents one such distribution of car loadings.

[Note; Car weights were converted to axle loads by adding to the car load the light weight of each platform.] The data shows the potential for a large percentage of current equipment to be overloaded.

The actual train operations were simulated in accordance with current BN operating rules on the Northern Region, which restrict TTX trains to a maximum of 70 cars, and twinstack trains to 21 five-unit cars. Although no stand-alone twinstack cars were then in service, the length, weight, and carrying capacity of the cars suggested a maximum train length of 90 cars, and this was used in the study. In addition, trains operating on the 2.2% grades of the Cascade Tunnel route were held to a maximum trailing tonnage without helpers. If helpers are added, the maximum train weight can be increased accordingly. Alternately, if the trains are operated via Pasco and Vancouver (see Figure 4), the severe grade limitation is avoided and train weight can likewise be increased.

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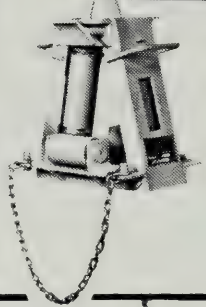

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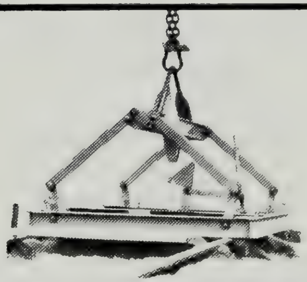

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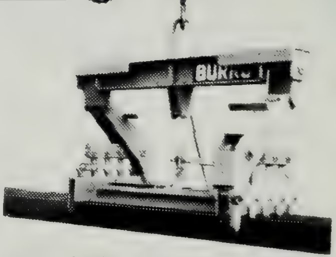

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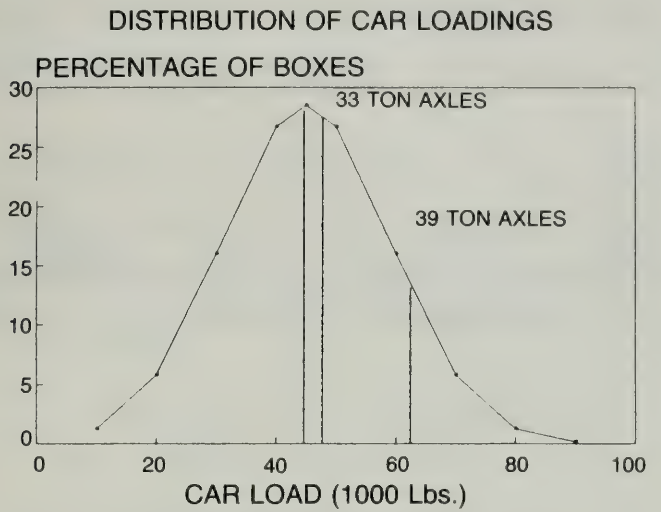

Figure 6

In this study, multiple combinations of train length and weight were simulated for the four different car types and three different route/operation combinations (Cascade Tunnel route with and without helpers and the Pasco Route without helpers.)

Track and Structures Analysis

Track costs were generated within the RECAP program by reference to a cost matrix generated from a series of TMCOST model runs (4). Although several of these matrices were developed for use in RECAP, these did not permit analysis of the cost impacts of incremental changes in axle loads. In addition, the 125-ton car costs (UNITH matrix) appear to indicate a linear increase from the 100-ton level, while published research strongly supports an assumption of nonlinearity. Since one objective of this study was to determine an optimum axle load rather than estimate the costs of one or two specified axle loads, it was necessary to develop an alternative costing methodology.

Track Costing Methodology

The 100-ton cost matrix within RECAP appeared to be a reasonably accurate predictor of the track maintenance costs associated with 100-ton cars, and thus served as a starting point for developing an alternative methodology. The approach used was that of developing an axle load dependent **"damage factor"** that increased the relative damage (and consequently cost) associated with increasing axle loads. This damage factor was then used as a "multiplier" to the 100-ton axle load costs to determine the costs associated with higher axle load traffic.

To determine such a damage factor, the sensitivity of each of the three major track components—rail, ties, and the ballast/subgrade—was examined. Each of these components is affected by several different degradation (damage) mechanisms, which are in turn related to axle load .

A literature search and supplemental analysis was undertaken to identify the range of variation in the damage relationships between the component damage categories and wheel loads. These damage factors were of the form:

$$F = (P/33{,}000)^n$$

where, for each deterioration mechanism:

F = calculated damage factor
P = actual wheel load
n = damage exponent

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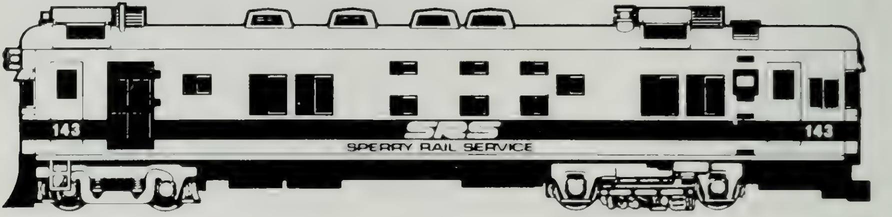

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For rail, the three primary degradation mechanisms are surface fatigue, internal fatigue, and wear. Surface fatigue and internal fatigue will increase exponentially with axle load. Wear, which is life-limiting principally on curves, is also dependent on axle load. A separate damage factor must be developed for each to quantify the impact of heavier axle loads.

Each of these three mechanisms was examined individually. For internal rail fatigue, the "worst" exponential value was that estimated by D. H. Stone of the AAR (5), who predicted a fourth power relationship between defect development and axle load. Marich (1) estimated a range from 2.0 to 3.3. The higher value was found on standard carbon steel rails installed before Mt. Newman Railway began a program of profile grinding. With the use of "clean" steels in more recent rail installations, a program of profile grinding, and the installation of premium rail on curves, the value for the damage factor is now believed to lie near the lower 2.0 figure. Other estimates have ranged from 3.3 to 4.0 for standard (non-premium) rail steels. Thus, for this study, the range for internal fatigue factor was assumed to lie in the range of 3.3 to 4.0 with an "expected" value of 3.3.

The literature shows an exponent of 3.0 to relate wheel load to fatigue on rolling contact surfaces. However, Clayton and Hill (6) identified a lower limit of 1.8 under certain conditions of load and lubrication. The range is thus from 1.8 to 3.0, with a value of 2.5 defined as the expected value for surface fatigue.

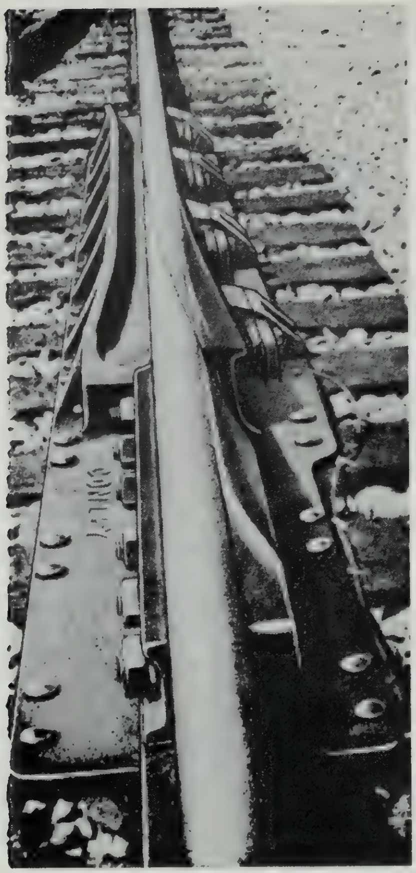
Wear has historically been treated as primarily linear with load. However, Marich (1), the Canadian Institute of Guided Ground Transport (7), and Reiner and Staplin (8) all have found a more-than-linear relationship. Thus, while the range of values lies between 1.0 and 2.0, on the basis of this recent research the expected value has been determined to be 2.0.

The distribution of these three mechanisms is generally related to the distribution of curvature and traffic on the track under study. Thus, internal fatigue is traditionally considered to be dominant failure or replacement mechanism on heavy-tonnage tangent track while wear dominates on curved track and on medium-tonnage, high-speed trackage (such as that used primarily by passenger trains or intermodal trains). **However, a review of Burlington Northern rail defect data suggested a larger role for wear and a much smaller one for internal fatigue in determining the replacement of rail.** BN personnel credited their aggressive rail grinding program for reducing the incidence of TDs, which appears consistent with recent research into profile grinding (9). They also felt that, between the grinding and the heavy traffic, metal loss from the rail head (wear) was the primary cause for rail removal from track. Based on this information, the relative weighting of the three failure modes were wear accounting for 65% of rail removals, surface fatigue (such as spalling and corrugation) for 10%, and internal fatigue only 25%.

Mechanical wear was the primary life-limiting mechanism for wood ties on the heavy-tonnage routes analyzed. Both Talbot (10) and Hay (11) consider tie life to be linearly related to load. However, Reiner (12) and CIGGT (7) indicated a non-linear relationship with exponent values ranging from 1.0 to 2.0. Noting North American experience with severe crushing of ties on curves in unit-train territory, a conservative exponent value of 2.0 was defined for wood ties.

For ballast/subgrade deterioration and the corresponding degradation of track geometry, traditional beam-on-elastic foundation analysis consider loading (and deflection) to be linear with axle load (11). There are, however, two exceptions. CIGGT (7) finds a square relationship, even with a good subgrade. Selig (13) and others agree on a grossly non-linear relationship between load and subgrade deformation on "poor" subgrades. The definition of poor subgrade is less than rigorous, however. While track supervisors know where their geometry problems are, the size and severity of the problem is extremely difficult to quantify.

Interviews with Burlington Northern Assistant Superintendents — Maintenance resulted in the following quantifications. Poor subgrades tend to deteriorate quite rapidly. Based on available evidence, 10% of the route being studied was assumed to have lesser subgrades, with an n value in the



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range of 3 to 7. The rest of the route, with good subgrade conditions, was assumed to fall in the range of n = 1 to n = 2. Expected values for poor and good subgrades are, respectively, n = 6 and n = 1.

Table 3 shows the resulting damage exponents using the best, expected, and worst case values.

Table 3: Ranges for Damage Factor Exponents

	Best	**Expected**	**Worst**
• Rail			
—Wear	1.0	2.0	2.0
—Surface Fatigue	1.8	2.5	3.0
—Internal Fatigue	3.33	3.33	4.0
• Ties	1.0	2.0	2.0
• Ballast/Subgrade			
—good ballast	1.0	1.0	2.0
—poor ballast	3.0	6.0	7.0
• Composite Totals	1.54	2.2	3.06

These values were combined in a logarithmically weighted average to derive a single n value that related track maintenance cost and axle load. Table 4 presents weightings and n values for the various subcomponents from which expected value for the composite track damage factor was produced.

Table 4: Final Track Maintenance Exponents

Component	**% of Total Maint. Costs**	**Load Ratio Exponent n**
Rail	.70	
—wear (65%)		2.0
—surface fatigue (10%)		2.5
—internal fatigue (25%)		3.33
Ties	.10	2.0
Ballast/subgrade	.20	
—good subgrade (90%)		1.0
—poor subgrade (10%)		6.0
Logarithmically Weighted Average n		2.20

Application of the weighted average best, expected, and worst n values through the wheel load equation for "damage" yields the damage factors shown in Figure 7.

These damage factors were then used to weigh the base 100-ton RECAP determined costs of track maintenance. However, the track maintenance costs calculated by RECAP using the 100-ton car matrix are built up from engineering models of track damage. Estimated track damage is turned into expenditures by calculating the cost — at industry-average prices — of the labor and materials required to perform the work the models indicate is necessary. Thus, these models calculate what work the railroad hypothetically should be doing. These levels of expenditure, however, do not bear any necessary resemblance to a railroad's actual expenditures. This is a common criticism of engineering-based models.

In this analysis, to insure that the calculated track maintenance costs were close to correct, the costs produced by the base 100-ton car were compared to Burlington Northern's actual average annual expenditures over a three year period for the line segments involved in the study (and the allocation of

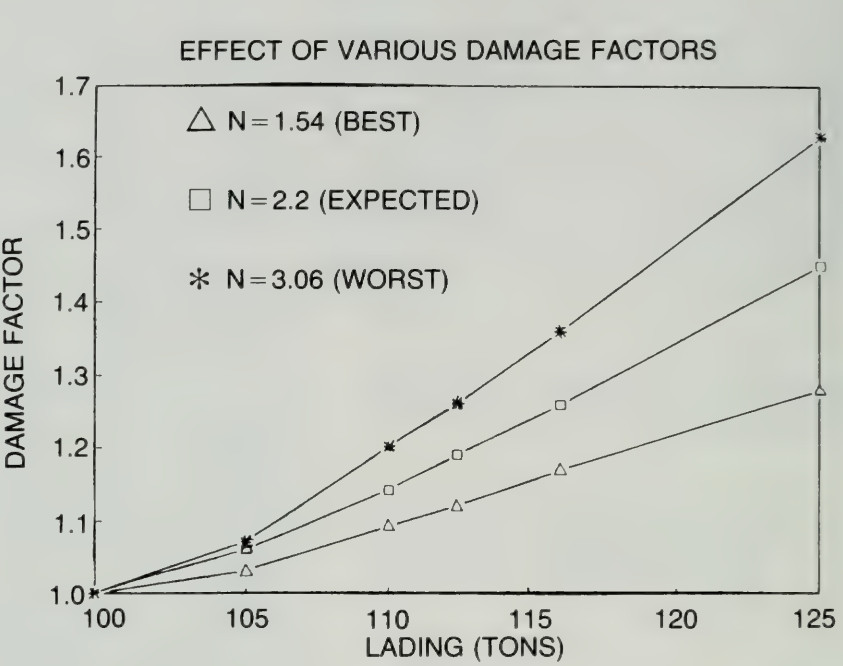

Figure 7

costs for the specific traffic analyzed). For the case of the coal movement, the RECAP calculated costs were found to be 86.5% of BN actual costs. Therefore, in order to relate the RECAP costs with BN actual costs, each of the individual RECAP cost elements was inflated by a factor of 1.156. A similar analysis was performed for the intermodal movement. In that case, a factor of 1.964 was required to match RECAP calculated costs with BN actual costs.

Bridge Maintenance and Repair

An area of major concern in this analysis was the cost of upgrading bridges to carry heavier loads. RECAP does not include a methodology for costing bridge maintenance and rehabilitation, but it was felt based upon the Australian experience that bridges represented a potentially major area of expense.

The operation of heavier trains can be expected to affect bridge costs in three ways. **First and most directly, ties on open-deck bridges may have to be replaced more frequently.** Since the same degradation mechanism applies to bridge timbers as to wood cross ties in ballast, and since bridge tie replacement is included in the previously detailed maintenance costs, any additional bridge timber maintenance expense has been captured in the inflation of track costs previously described.

Second, heavier loads may necessitate structural repairs to strengthen bridges and increase their load ratings. Alternatively, impact loads can be lowered by reducing speeds. Burlington Northern's Structural Engineering Department was asked to evaluate the impact of increased wheel loadings on bridges on the two study routes. They calculated that most bridges would be able to sustain the heavier loadings without quantifiable damage. Two groups of bridges were identified as problems, however. The first group consisted of bridges with load limits so close to the propsoed maximum wheel loads that slow orders would be required to limit dynamic impacts. These slow orders were

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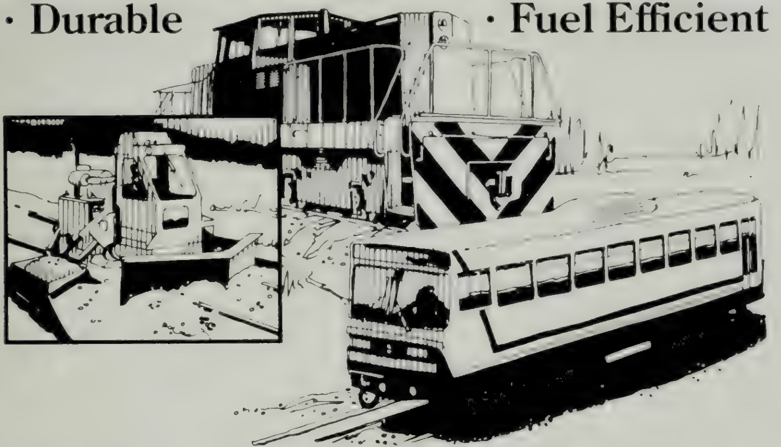

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incorporated into the simulation track files, and all heavy car simulations were run with the slow orders. No operating problems were encountered.

The second groups of bridges has riveted rather than bolted floor beam hangers. Replacement of the riveted hangers with bolted fasteners — a desirable upgrade in any case — was essential for the proposed heavier loads. The discounted cost of this bridge upgrade was introduced as an additional bridge maintenance cost for the heavy axle load cars.

The third concern is the reduction in bridge fatigue life that might occur with heavier loadings. Bridge life can be measured in number of load cycles; the heavier the load, the fewer the cycles until the bridge fails and must be replaced. A greatly over-designed bridge may never fail, since the cyclic loading may be below the endurance limit for the bridge steel. However over designed the Burlington Northern's bridges may have been, many are more than seventy-five years old and almost certainly subject to fatigue.

Noting the standard relationship between stress (S) and cycles to failure (N) for structural steels:

$$\log (S) = -a(\log (N)) + b$$

where a is the slope of the stress/cycles (S/N) curve, as shown in Figure 8. (Note that $a = .33$ ($1/a = 3$) for standard structural steels, as used in railway bridges [13]). The stress value S, in turn, is directly related to the loading P. For linear elastic structures, such as steel bridges, the stress can be assumed to be linear with load. Noting these equations, the relationship between the loading and life (in cycles to failure) for the different car types (and axle loadings) was taken to be:

$$(P/P_{ref})^{1/a} = (N_{ref} / N)$$

where:

a = the slope of the AREA bridge steel S-N curve
(a = .33; see Figure 8)

P_{ref} = 33,000 (100 ton car)

P = actual load [i.e., 35,750 (112.5 ton car)]

and N_{ref} = the expected life of the bridges at reference axle loads

A recent survey of one of the specific bridges (on the coal study route) showed that certain structural members had only 25% of their fatigue life remaining (15). Most or all fatigue damage was assumed to have occurred since 1970, when 100-ton cars first became common. From this data, a reference life of 24 years was inferred for those bridges close to their load limits under existing traffic.

Using this 24-year reference life for current traffic, S/N analysis was used to calculate the remaining life of these bridges, at the various axle load levels.

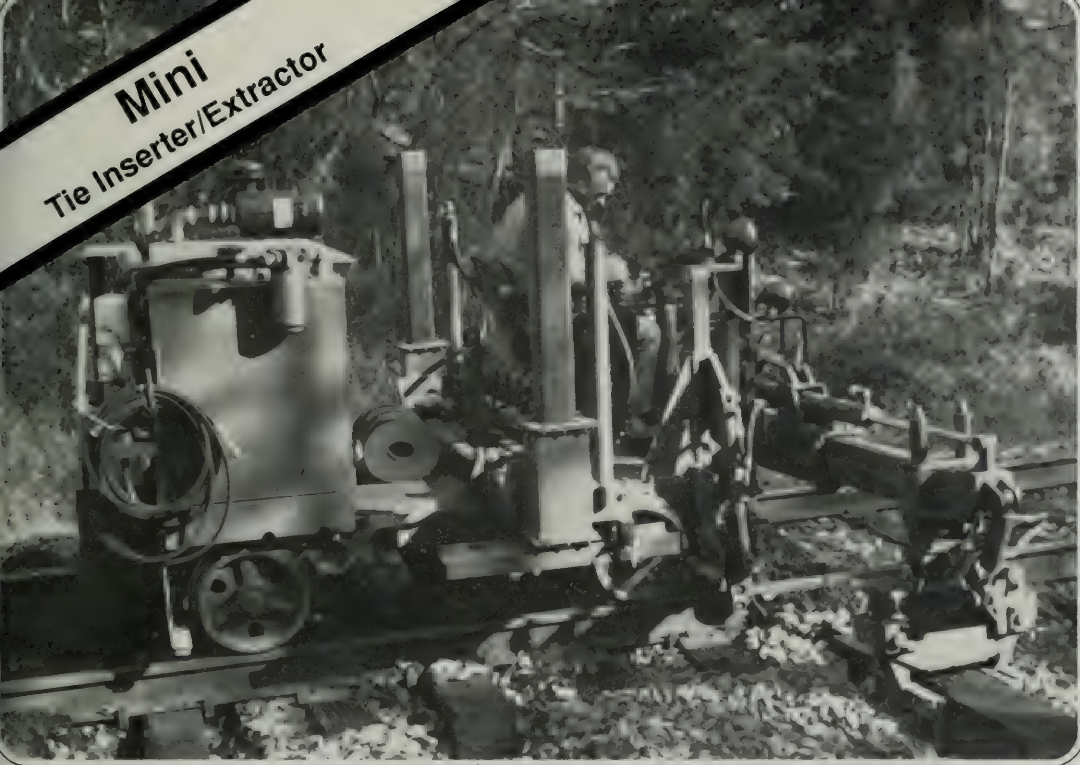
For those bridges that had E-ratings that exceeded the loading imposed by 112.5 ton cars by only a moderate amount, a 40-year reference life was assumed because of their substantial excess capacity under current loadings. S/N analysis again was used to estimate lives under the proposed heavier loadings.

To estimate the cost impacts of this reduction in life, an average replacement cost per foot for each of the major types of bridges was obtained from Burlington Northern's System Bridge Engineering Section.

A complete list of bridges on the study routes was prepared which **divided the bridges into four categories.** These were:

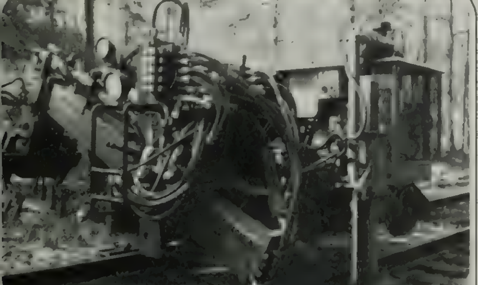
- Bridges felt to be adequate to accommodate the heavier loads

Mini
Tie Inserter/Extractor



Features 53" boom stroke and maximum reach of 93-3/4"

Fully proportional electronic joysticks or optimum control during tie operations.



Unit in raised position ready to "walk" off track.

- Joystick controls give precise proportional contro over tie extracting/inserting operations.
- Boom lifts 25 degrees above horizontal — 15 degrees below horizontal. Slew movemen is 15 degrees to accomodate slewed or switch ties and for tie change-out.
- Optional "walking beam" set off clears the right-of-way in six minutes, and will operate or inclines up to 30 degrees.
- Removes/inserts up to 55 ties per hour.
-

Frame:	Welded tubular steel	
Length:	124"	(3,150 mm)
Width:	107"	(2,718 mm)
Height:	83"	(2,108 mm)
Weight:	7000 pounds	(3,175 kg)
Wheelbase:	60"	(1,524 mm)

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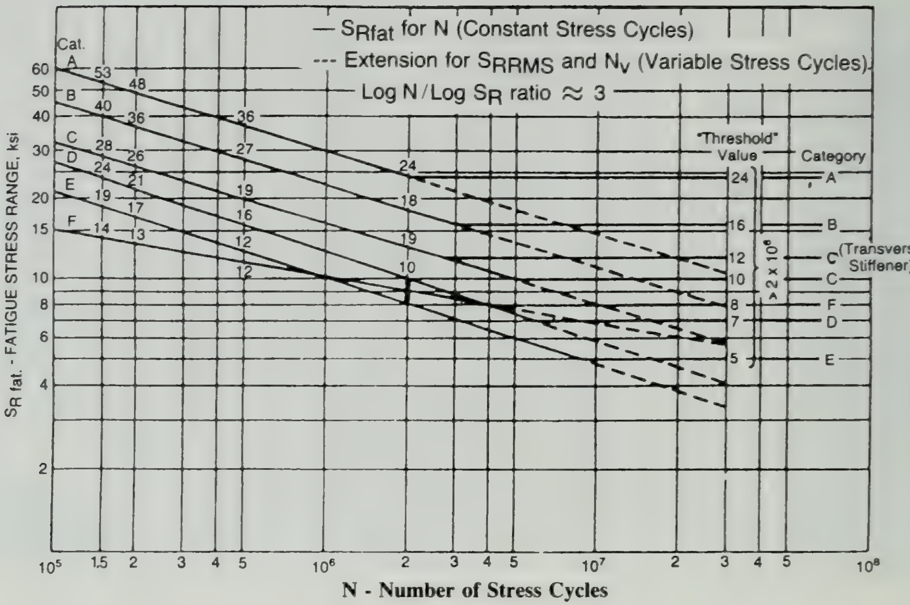

Figure 8
S_R-N Curves for Bridge Steel

- Bridges with E-ratings below the loadings imposed by the heavier cars, and requiring either replacement or slow orders. (Bridges in this category were assumed to have only 25% of their fatigue lives remaining, and, as a result, a 24-year reference life).
- Bridges requiring replacement of riveted with bolted hangers in order to accommodate the heavier cars.
- Bridges with E-ratings less than 5% above the level required to accommodate 112.5 ton cars (Bridges in this category were assumed to have a 40-year reference life).

It is assumed that when a bridge reaches the end of its fatigue life, it will be completely replaced. This is a conservative assumption. Replacement bridges, which will be designed for 125 ton (or heavier) loads, are assumed to have an eighty-year life.

Using the projected cost of the bridge replacement and the calculated life of the bridge, for each axle load category an annual cost of bridge replacement was calculated together with the incremental cost of operating heavier cars (which is the difference between the annual costs between present loading and increased load levels). Note: the only cost properly chargeable to heavier cars in this comparative analysis was the incremental cost due to the shortening of bridge life from current levels. It is this incremental annual bridge cost which was charged to the heavy axle load vehicles.

For the case of the intermodal equipment, which had a distribution of axle loads, as noted previously, bridge costs were calculated for each of five axle load categories (33, 36, 39, 42, and 45 tons) and combined in accordance with the actual percentage distribution of these axle loads in the movements (as determined by the statistical analysis of container weights and distribution). Once again, the incremental annual bridge cost was then charged to each set of equipment as a function of its capacity and axle loading.

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Equipment and Car Maintenance Costs

Car equipment and maintenance costs were calculated using the RECAP Economic Routine. The ownership cost per hour was calculated using the purchase price, discount rate, and economic life of each type of equipment. This cost was then applied to the simulated hours of operation to obtain the ownership costs. The maintenance cost was calculated for the number of miles operated by each piece of equipment through the use of a per-mile maintenance charge.

To account for the effect of increasing loadings of existing cars (where there will be a significant increase in maintenance costs as the design loadings are exceeded) a damage factor approach was used. These damage factors increase the per-mile car maintenance cost to reflect the stresses imposed by heavier loads.

The equation takes the same form as the equation used to increase track costs:

$$F = (P/33{,}000)^{m}$$

where, for each deterioration mechanism:

F = calculated damage factor
P = actual wheel load
m = damage factor

Damage factors have been calculated for major car components and subcomponents in the same manner as track costs and are presented in Table 5.

Table 5: Car Maintenance Cost Factors [15]

Component	% of Total Maint. Costs	Load Ratio Exponent m
Wheels and Axles	.58	
—wear (50%)		1.0
—shelling (20%)		3.33
—discolor (20%)		1.0
—thermal damage (9%)		2.0
—journal fatigue (1%)		9.0
Air Brakes	.03	
—shoes (80%)		2.0
—system (20%)		1.0
Trucks	.15	2.0
Bearings	.13	3.33
Couplers	.04	4.1
Car Body	.07	3.33
Logarithmically Weighted Average m		2.19

During the course of this study, an economic assessment was also made from the addition of bearing adapter pads to the intermodal double-stack cars for enhanced vertical ride performance.

Derailment Costs

Derailments, while infrequent, can be very expensive. In order to introduce the cost of derailments into this study, an annual cost of derailments on the routes under study was obtained from BN data and allocated to the movement under study as a percentage of traffic. These derailment costs were then increased to reflect axle load increases in a manner similar to the methodology of increasing track and equipment costs. The equation takes the form:

$$\text{Derailment cost [heavy axle load]} = \text{derailment cost [base]} \times F$$

where F is the inflation factor derived from the track and structural analyses through the equation:

$$F = (P/33{,}000)^N$$

and N is the appropriate exponent for either track or equipment costs.

Since previous estimates of increased track and equipment maintenance costs assumed a shortening of component lives and an increase in maintenance activity, this estimation of derailment costs may represent double counting. A railroad may, for example, increase inspection frequency to avoid additional derailments due to broken rails, or institute a bearing inspection and changeout program. These procedures are part of the increase in costs already assigned to the operation of heavier cars; however, they will help to limit derailments.

Implications from the Heavy Axle Load Studies

Given the distinctly different nature of these two studies, the results and inferences will be presented separately.

Coal Movement

For the coal movement, three areas of savings were identified. The first was the result of operating heavier trains (as opposed to heavier cars). The second area of savings results from the increase in car weight. Although both car and track maintenance costs increase as car weight increases, net savings can result simply from an increase in car weight. The third area of savings comes from optimizing motive power assignments.

The substantial economies available from increased train size suggested that the optimum train consist would be one of the heaviest trains simulated and that was determined to be the case.

Figure 9 combines several different simulation runs on one graph to show the process by which the minimum-cost consist was reached. Several factors are at work. The first two data points show the effect of increasing car (and thus train) weight while holding train length constant. This would be the preferred strategy if siding length placed a constraint on train length. If siding length is not a constraint, the third point shows the cost of a maximum-tonnage train of 101.5 ton cars. Note that it is less costly to operate than any of the fixed train length cases. But holding train weight at or near the maximum of approximately 17,100 tons while increasing car load continues to **produce economies up to the maximum feasible load of 112.5 tons** (fourth and fifth points). Note, however, that at the 125 ton car loading, on this route, the costs start to move back up. [This is partially due to a large bridge cost that is incurred at the 125 ton axle load level.]

The total cost curve decreases to a minimum for the 120 car, 112.5 ton per car train. This is the optimum (least cost) alternative for the route studied. The 112.5 ton load is also the practical maximum, determined by coal density and car volume. The curve then turns up to the 125 ton cars. **The overall transportation benefit amounts to a projected annual savings of 5.2%.** However, the total cost at 125 tons per car is still less than for the base case consist. Train length/car weight combinations less than the optimum consist, such as the 115-car, 110 ton per car consist, will still produce significant savings over the base case although not as large as those for the optimum case. [It should be noted here that equipment purchased new, and designed for greater-than-100-tons loads, would certainly not be subject to the extreme increases in car equipment maintenance costs produced in this analysis.]

Table 6 lists all consist/motive power combinations simulated and their overall relative costs. Figures 10, 11 and 12 present the relative costs for Track Maintenance, Equipment, and Operations. The trends can be clearly seen; track costs (Figure 10) always increase with axle loads, while equipment (car and locomotive) costs (Figure 11) decline with increasing train weight in most cases. While total

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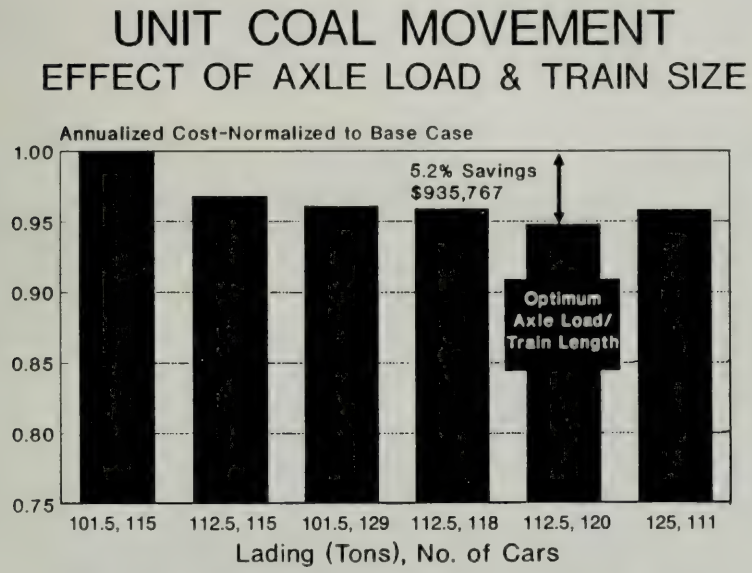

Figure 9

car maintenance costs must increase as axle loads increase, since savings in car mileage are linear while car maintenance costs increase exponentially, the saving in locomotive miles resulting from the smaller number of annual train trips is sufficient to offset the increase and lower total annual equipment maintenance and ownership costs.

Table 6
Results

Consist*	No. of Cars	Car Weight (Tons)	Loco Assign	Relative Costs	Savings
G1	115	101.5	BASE	1.000	—
H1	115	112.5	BASE	0.967	3.3%
I1	129	101.5	BASE	0.961	3.9%
L1#	120	112.5	BASE	0.948	5.2%
M1	118	112.5	BASE	0.959	4.1%
O1	111	125.0	BASE	0.958	4.2%

# Represents optimum "least cost" case.

As expected, operating costs (Figure 12) decrease with increasing axle load and/or train size.

Intermodal Container Movement

The results of the heavy axle load intermodal container movement study are presented in Figure 13 for one of the studied routes. As can be seen in this Figure, the **TTX 89-foot flatcars are a very expensive way** to move ocean containers when compared to double stack equipment. **Articulated**

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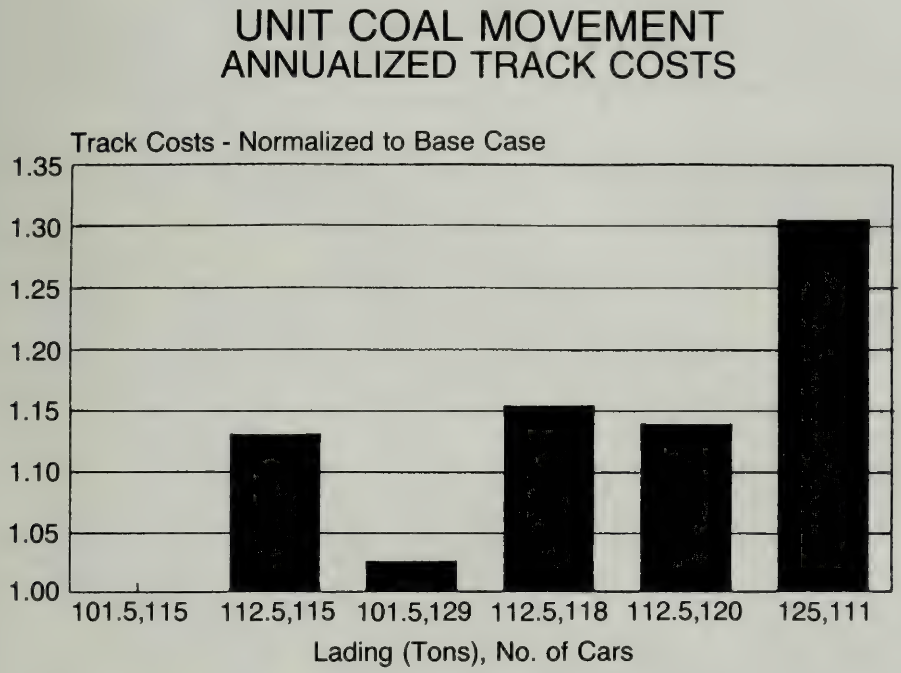
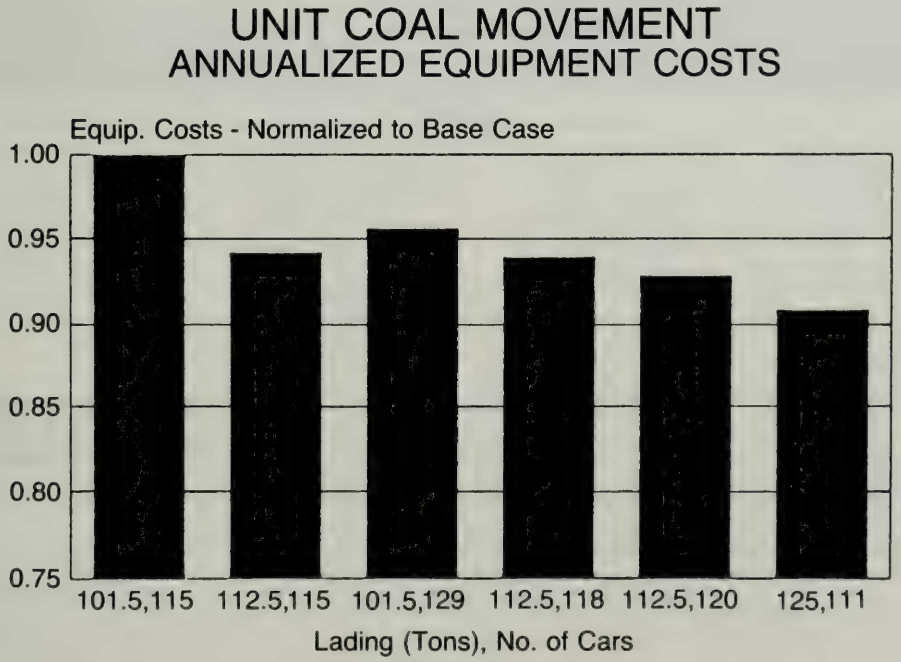

UNIT COAL MOVEMENT
ANNUALIZED TRACK COSTS
Track Costs - Normalized to Base Case
1.35
1.30
1.25
1.20
1.15
1.10
1.05
1.00
101.5,115
112.5,115
101.5,129
112.5,118
112.5,120
125,111
Lading (Tons), No. of Cars

Figure 10


UNIT COAL MOVEMENT
ANNUALIZED EQUIPMENT COSTS
Equip. Costs - Normalized to Base Case
1.00
0.95
0.90
0.85
0.80
0.75
101.5,115
112.5,115
101.5,129
112.5,118
112.5,120
125,111
Lading (Tons), No. of Cars

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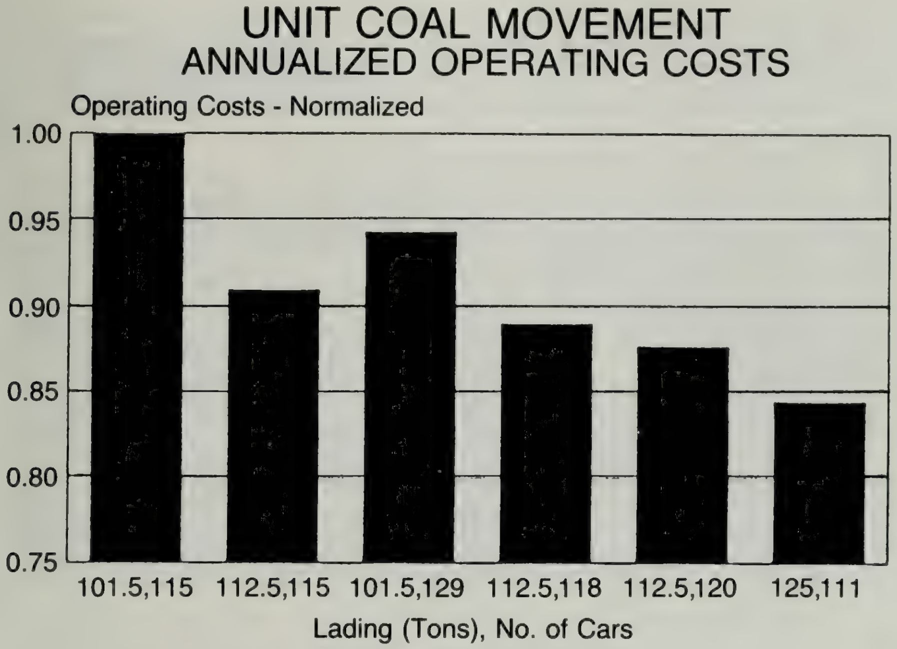

Figure 12

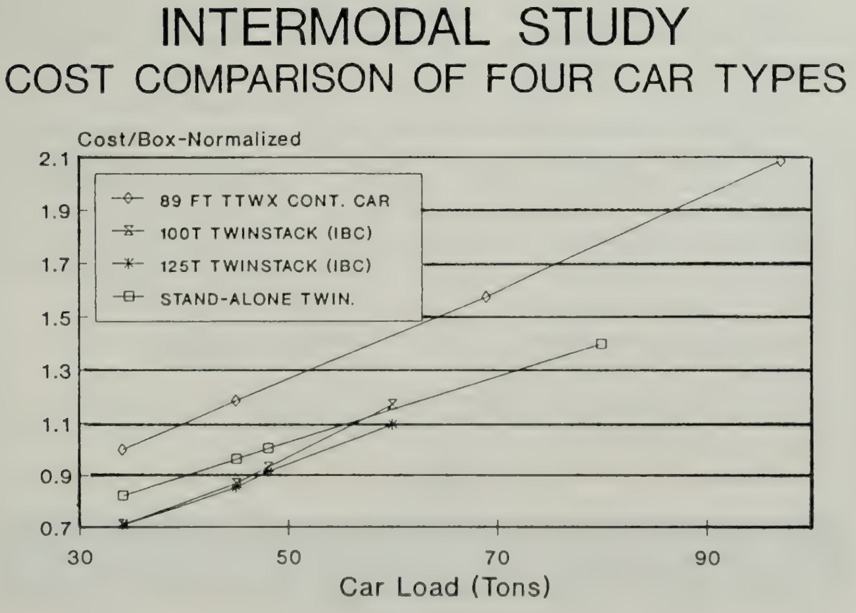

Figure 13

double-stack cars, while relatively expensive per car mile, are by far the least expensive cars to operate on a per-container basis. Only at very heavy average car loadings does the lesser cost per container for the doublestack equipment approach that for the stand-alone car.

At a theoretical difference of $200 per box for double-stack verus conventional equipment and 150,000 containers handled per year, a projected $30 million can be saved each year. The 125-ton IBC car, which incurs no car maintenance penalty at car loads of up to 60 tons, is less costly to operate than the stand-alone car at all car loads simulated. It must be noted here that given the mix of container weights, and the current loading process, 100 Ton twinstack equipment can experience almost identical loadings to the 125-Ton equipment, but without the benefit of increased car body strength and larger wheels and axles. Furthermore, it was noted that even with 125 ton equipment, overloading was still possible, given the distribution of containers that are carried.

Closing Observations

There is an economic benefit to increasing axle loads beyond the generally accepted 100-ton (33 ton axle load) maximum at BN. There are some very specific benefits to be realized from increasing net load by increasing car weight. They include:

—A reduction in car miles and car ownershp requirements

—A reduction in train resistance due to the smaller number of axles required for an equivalent train weight

—An improved net-to-tare ratio, which reduces fuel costs

—An increase in train weight without adverse operational impacts resulting from inadequate siding capacity.

However, while in general heavier axle load trains are more economical, general conclusions should not be drawn about optimum car weight or optimum train size without evaluating route characteristics. The results reported apply predominantly to the two routes studied. Specific route characteristics, such as grade, curvature, siding lengths, and bridge conditions will determine the actual optimal train size and weight.

New equipment designed for heavier loads will also avoid the severe exponential increase in car maintenance costs and derailment costs assumed in these analyses, and may produce savings at car load levels higher than 112.5 tons payload (36 ton axle loads). Quantification of the savings available with the operation of coal equipment in newly-purchased 125-ton cars was beyond the scope of this study and is being developed in BN's ongoing research studies.

In summary, Burlington Northern Railroad believes that a thorough, yet conservative analysis of the implications of increased axle loadings has been accomplished weighing all of the critical elements with the following findings:

- Significant savings can be attained by increasing the axle loads of existing coal cars to the 35 to 36 ton axle load limit (on the order of 5% reduction in annual operating costs).

- Significant savings can be attained through the use of 125-ton double stack container cars versus conventional flat cars (on the order of $100 to $200 per container on long route movements, depending upon route characteristics and $15-30 million total savings).

REFERENCES

1. Marich, S. and Maass, U., "Higher Axle Loads are Feasible—Economics and Technology Agree," *Fourth International Heavy Haul Conference,* 1986.

2. Burlington Northern Task Force, "Heavy Axle Load/Train Size Costing Study for Unit Coal Train Service," (Confidential), BN Report 5.07—April 25, 1988.

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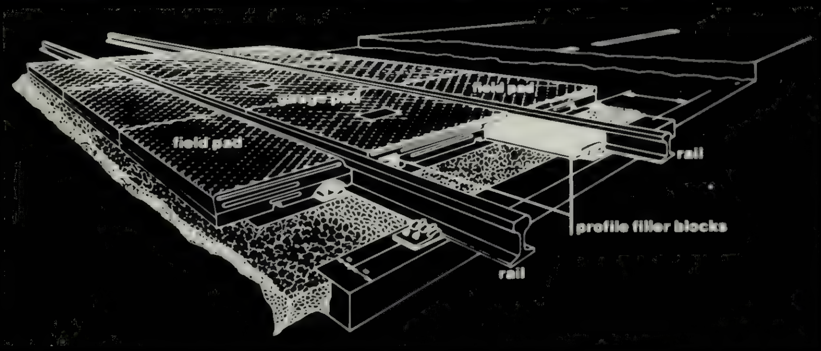

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3. Burlington Northern Task Force, "Heavy Axle Load/Train Size Costing Study for Intermodal Traffic," (Confidential), BN Report 3.07—December, 1988.
4. Smith, Michael. *Rail Energy Cost Analysis Package: The Route Perspective.* Association of American Railroads, 1987.
5. Stone, D. H., "Comparison of Rail Behavior with 100-ton and 125-ton Cars," Association of American Railroads Report R-405.
6. Clayton, P. and Hill, D. N., "Rolling Contact Fatigue of a Rail Steel," *Contact Mechanics and Wear of Rail/Wheel Systems II,* Univ. of Waterloo Press, 1987.
7. Raymond, G. P., et. al., "Analysis of Rail Track Structures (ARTS) User's Manual," Canadian Institute of Guided Ground Transport CIGGT No. 79-20, April, 1980.
8. Reiner, I. A. and Staplin, D. E., Empirical Rail Wear Model, Association of American Railroads Report WP-104, March 1983.
9. Worth, A. W., et. al., "Prolonging Rail Life Through Grinding," Third International Heavy Haul Conference, Vancouver, 1986.
10. Talbot, A. N., et. al., "Stresses in Railroad Track—The Talbot Reports," American Railway Engineering Association, 1980.
11. Hay, William. *Railroad Engineering.* John Wiley & Sons, 1982.
12. Lewis, W. F., AAR Cost Analysis Organization Subcommittee; Economics of 80 vs. 100 Ton Cars, Final Report of Engineering Panel, July 1981 [Exhibit; Reiner report].
13. Selig, E. T., et. al., "A Theory for Track Maintenance Life Prediction," Dept. of Transportation Report, DOT/RSPA/DPB-50/79/22, Washington, DC, August 1979.
14. American Railway Engineering Association, "Manual for Railway Engineering," Chapter 15, Steel Bridges, 1987.
15. Hazlet and Ertal, Inc., Bridge Inspection Report (Burlington Northern Railroad, 1987).
16. Moyer, J. "BN Optimum Loaded Car Weight/Train Size Project," Report to Burlington Northern Railroad, August 1987.

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BALLASTED TRACK FOR RECORD SPEED TEST

By: M. P. Roumeguére*

I. BACKGROUND

In September 1981, The French Railroad opened to commercial service the first 260 km/h** high speed line on a section of a new route between Paris and Lyon. In February, seven months before this opening, a new record speed of 380 km/h was reached on this line. Two years later, the entire South East line was opened to commercial service at an increased speed of 270 km/h. In September 1989, two thirds of the new Atlantic Line opened at a commercial speed of 300 km/h. During the previous year, several test runs had taken place both on this new route, up to a speed of 390 km/h, and also on the South East Line (which was at that time in commercial service), up to a 409 km/h figure, slightly better than the German ICE Record Speed. At the beginning of December 1989, a new series of test runs brought this maximum speed to the 482 km/h value, 180 km/h more than the actual commercial speed.

As this short relation of events shows, these consistently repeated test runs do not solely aim at breaking records. They are, in fact, the most visible part of a well defined development plan for French rail high speed techniques involving the whole TGV system. Among other factors, this development plan calls for higher speed in the fututre, for which long time testing will be required. So, even if the

TGV Atlantic train now in regular service at 300 kph with 10 cars and locomotive at each end. 482 kph test run had same basic type of equipment, but only four cars between the locomotives.

---

*Directeur de L'Equipement, Societé Nationale des Chemins de Fer Français

**This article is presented in metric units. Convenient conversion factors are:

100 km/h = 62.136 mph
1 km = 1000 m = 3280.8 ft.
10 mm = 1 cm = 2.54 in. (exactly, by definition)
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actual commercial speed is set below the official allowance by some 10% (270 km/h on the South East Line compared with a possible 295 km/h, 300 km/h on the Atlantic Line compared to a possible 330 km/h), this increased speed is not sufficient to make the necessary investigations in the higher speed range of 350-400 km/h.

Apart from certifying the ability of the system at a speed at least 80-100 km/h higher than the commercial service, these tests allow continued testing of vehicles on dedicated sections of the new lines at speeds of 350 km/h on the South East Line and probably 400 km/h on the Atlantic Line as a current practice, i.e. with only limited maintenance restrictions.

Description of the Test Sites

On the South-East TGV Line

Following the high speed tests in 1981, a section of the line of around 80 km is currently opened for high speed tests in the range of 320 to 345 km/h, from Lacour D'Arcenay to St. Florentin on track No. 2. The middle part of this section, having a minimum curve radius of 10,000 m can even be used for higher speed tests between km posts 177 and 148, the speed restriction on the surrounding curves (4500 and 6000 m) being 345 km/h. Maximum slope is 3.5% and it should be noted that a total of five switches are to be crossed on the whole section, three of which near the point of maximum speed at Pasilly.

On the Atlantic Line

During the alignment design of this line, it was possible on the South-West branch between Courtalain and Tours to adopt rather large curve radii, bearing in mind that the commercial speed could be increased in the future on both tracks. From Courtalain (km post 130) where the western branch of the line separates, to km post 148, minimum curve radius is 6,000 m. Then from km 148 to 178, this minimum radius increases to 15,000 m, and is still 10,000 m until km 196. This section thus offers at least 30 km of very high speed potential. In fact, to be able to reach the maximum possible speed, test trains must depart from a siding in Dangeau (km 114), and have to cross the Courtalain turnouts on the direct branch at more than 350 km/h before entering the test section by itself. Both tracks can be used for those tests; maximum gradient is limited to 2.5%. It should be noted that on this test section the maximum speed is to be reached between km 166 and 167, that is on a viaduct over the Loir river.

Track components are the same on the Atlantic TGV Line and on the South-East: UIC 60 rail with the Nablo resilient fastenings incorporating a 9 mm grooved rubber pad, twin-block reinforced concrete sleepers spaced at 60 cm intervals and laid over 30 to 35 cm of hard ballast on a well graded and compacted platform. This now well known superstructure is the current SNCF standard on high speed lines.

II. TECHNICAL CONSTRAINTS ASSOCIATED WITH HIGH SPEED TESTS

Of course, such tests should be non-destructive ones, and the opening of such a new speed range raises several questions long before the first run. Among them, and apart from the necessary power transmission and braking requirements, the most important are related to:

- The wheel/rail interaction and forces, and more generally stability of the vehicle,
- The behaviour of specific structure elements such as turnouts, bridges . . .
- Both above subjects concerning the track superstructure by itself,
- The pantograph/catenary mechanical and aerodynamical interaction.

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Wheel/Rail Interaction and Forces

Lateral forces exerted by the trucks on the track depend on several factors:

- The quasi-static lateral force in curves, depending mainly on cant deficiency.
- The random component, depending in the case of the stable truck, on speed, cant deficiency and geometrical quality of the track. In the case of an unstable truck (an event which should be avoided) the lateral forces can reach rather high values depending on the damping characteristics which are normally introduced by the links between the truck and the body of the vehicle.

Vertical forces are strongly dependent on the geometrical quality of the track, both in the long wave and short wave domain of defects, and also on speed. The track must, in any case, be able to bear these forces, which should in turn be limited by the following considerations:

- On ballasted track, it is well known that the lateral resistance of the track structure in the ballast, which acts as a plastic deformation above a given level of side pressure, is limited. Practical values used by the SNCF for commercial service set this limit at the *PRUD'HOMME* value, $H_{lim} = 0.85 (10 + P/3)$, with P the axle load in Kilonewtons. In fact, this limit stands for a non-stabilized track with wooden sleepers, and since several measurements made in 1981, we know that on the large twin-block sleepers in use on the TGV lines, we can expect much higher values, at least for a TGV axle load of 170 kN:
    - 87 kN for a non-stabilized track
    - 126 kN for a track after dynamic stabilization.
- Another limit arises, above the *PRUD'HOMME* limit, when considering the derailment coefficient, i.e. the ratio of lateral and vertical wheel loads of the front external wheel of the truck. An usually accepted limit for this coefficient is $Y/Q = 1.2$, but of course no experience was at that time available on the effect of speed in this range. However the SNCF had already a long experience of 200 km/h practice, and eight years of 270 km/h service from which it was known that this ratio is scarcely trespassing 0.6 to 0.8 values with an excellent safety margin.
- As far as the vertical load is concerned, experience shows that only very high dynamic forces, often related to short geometrical defects in the rail, would deteriorate the track with the first sign being the cracking of sleepers. However this limit is very high, standing at some 200 kN vertical force on the rail seat, which means about double this value on top of rail. Of course, the minimum wheel load value is a direct component of the Y/Q ratio.

Specific Superstructures Constraints

During the tests, the train would have to cross:

- Turnouts and expansion joints or points where the track structure is somewhat different from the regular track structure, such as in a switch where the tongue rail is not strongly attached to the sleepers, and may have developed a higher level of vibrations.
- Bridges, by dynamic behaviour, were also to be considered as possible points of important track structure vibrations.

III. THE THREE PERIODS OF THE TEST SEQUENCE

As far as the track superstructure was concerned, these considerations led to the organization scheme of the whole test, also based on the previous very high speed test results of 1981 and on the numerous tests conducted from 1981 to 1988 in the range of 300-350 km/h. These previous tests had already provided:

- a rather complete survey of vertical and lateral wheel/rail forces in the 300 km/h domain, on a specific range of cant deficiencies and track geometry qualities

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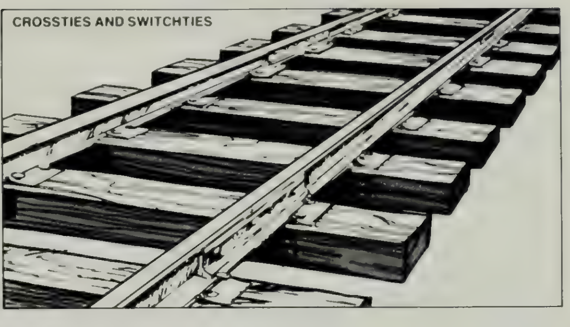
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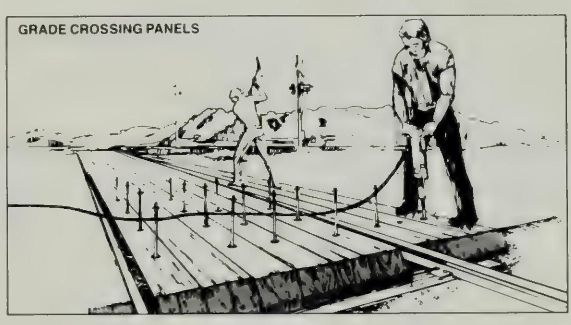
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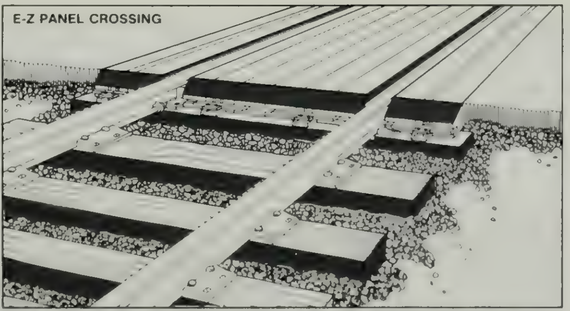

E-Z PANEL CROSSING

- a good correspondence between lateral accelerations of the trucks and bodies of the vehicles, and the wheel/rail forces, in the same conditions
- a first approach of the stability conditions of the trucks in the 300-380 km/h range

Hence the following test sequences, with a careful progression of the testing speed:

- A first series of test runs, with the aim of exploring more completely the 350-400 km/h domain in terms of:
    - behaviour of the vehicle body and trucks
    - behaviour of the pantograph-catenary system

These test runs were made on the South-East Line with a slightly modified TGV-PSE trainset, No. 88, with a composition reduced to 7 vehicles. A maximum speed of 408.4 km/h was then reached, a little more than the then world record of the ICE, and the overall results did not show any limitation for a further increase in maximum speed objective.

- A second series of tests was then organized in August 1989, immediately following the completion of the construction of the South-West Branch of the Atlantic line.

Two objectives were aimed at:

- As usual after such completion, and before the opening of the line to commercial service, a certification of the line in the 300-340 km/h range is necessary. As fine as the positioning of the track can be made in level and alignment during the construction works and controlled by the track recording coaches, a few points always need some small adjustments. It is one of the advantages of ballasted track to make these corrections rather easily with the usual tamping and lining machines, even following the first high speed runs on the line.
- A second aim of the test was to get even more information about the wheel/rail interaction on this particular section of track in the 370-390 km/h range, so as to bring about further work if necessary to bring the track geometry to the needed status, both for current track conditions and for special points.

  In fact, after several runs reaching a maximum speed of 390 km/h made by the Atlantic trainset No. 308, limited to ten vehicles, only a few adjustments were recognized as necessary for the track geometry. This shows the very high level of geometrical quality directly achieved by the construction works.

IV. THE DECEMBER 1989 VERY HIGH SPEED TEST

IV.1 Trainset No. 325

Among the Atlantic Line fleet, trainset No. 325 was chosen for this test. With only limited modifications, this trainset was still very similar to the basic series standard trains:

- First, composition of the train was reduced to six vehicles, two engines and four trailers, to enhance power and reduce aerodynamic drag
- Traction regulation equipment was set to give the necessary characteristics both for traction and braking. The traction motors themselves were the standard series motors used during the test at a much higher power than the normal 300 km/h setting: 13,000 kW compared to 8,800 kW for the whole train.
- In the mechanical domain, the diameter of the engines' wheels had been changed for 1,050 mm, and the reduction gears were given a 1.065 ratio.
- To enhance stability of the trucks, all wheels had been profiled to the classical GV 1/40 profile, and yaw dampers with friction characteristics installed.

- Aerodynamically, the rear pantograph only had been kept. The three others were taken off and had been replaced by fairings.

Following these modifications, the acceleration possibilities of the train on the test section were as below, without the need to use full power during all the acceleration phase:

from 0 to 400 km/h in 6 min. 25 sec over 22 km
from 0 to 440 km/h in 8 min. over 33 km
from 0 to 460 km/h in 9 min. over 40 km

IV.2 The On-Board Laboratory and Instrumentation

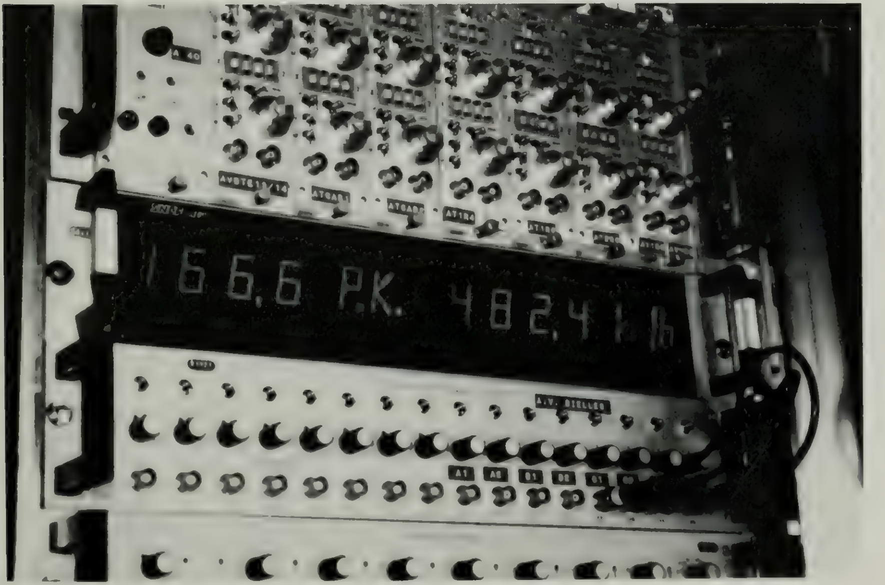
The first trailer coach incorporated a complete laboratory, surveying some 150 parameters which were necessary both for the control of the test and for future scientific analyses.

This instrumentation was mainly devoted to:

1. A dynamic mesuring unit for the control of:
    - stability of all the trucks in relation to the lateral accelerations
    - lateral and vertical wheel/rail forces exerted on the track by the front driving truck and by the second trailer truck
    - the dynamic behaviour of trailer and engine body through lateral and vertical accelerations

2. A pantograph-catenary measuring unit, with pantograph displacements and acceleration measurements including video survey

3. A measuring unit for several traction and running parameters such as:
    - electrical and thermal parameters of the traction equipment
    - thermal survey of the axle-boxes
    - mechanical control of some elements of the traction chain



Indicator on train showing attainment of 300 mph record speed.

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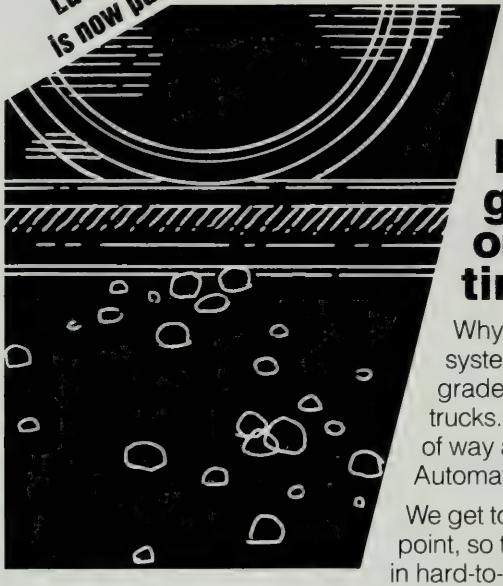

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IV.3 Site Measurements

Along the track some 200 parameters were also measured in different sites to control the following aspects:

1. Plain track behaviour

During the speed increase sequence, vertical and lateral loads on the rails were measured in the 15,000 m radius curve near the maximum speed point. This was to allow for comparison with the on-board measurements. In a second period of the tests, these measurements were completed by some investigative ones concerning stresses, displacements and accelerations in the track structure under fast moving loads.

2. Turnouts and expansion joints

Displacements and stresses were also measured in two switches in Droue and in the expansion joint of the Naveil viaduct, and on the guard rail of a fixed nose crossing in St. Amand, which we had to cross at more than 400 km/h.

3. Dynamic behaviour of the catenary

Was controlled both in the interlocking catenary mounting in Droue, and in the plain track near km post 166.

4. Dynamic behaviour of several rail bridges was controlled in the very high speed section of the track.

5. Environmental measurements including noise and vibration emission by the train were also taken near the maximum speed point. Also measured was the wind effect of the passing train at different distances from the track.

IV.4 The Test Procedure

Each day, the following sequence of events was repeated:

- Early in the morning, a classical train hauled by a diesel locomotive (electric locos allowed on the new lines being in very small number) opened the line at 80 km/h. It incorporated several measuring coaches including the Mauzin track geometry recording car, by which it was possible if necessary, to control any change of track geometry after the preceding tests. In fact, this measurement was not recognized as useful every day, with no movement of the track being observed.
- Then the test runs by themselves could take place on the test section. Three such tests were normally run each day, each of No. 325 runs being preceded by a run of a No. 308 trainset at 350 km/h. With the on-board measurement of lateral truck and body accelerations on the later, these records were playing the role of a track geometry control, by which any change in the track position would have been discovered.

The test runs by themselves were of course conducted with careful attention to some external parameters, such as rail temperature, wind, . . .

V. THE MAIN RESULTS

V.I The Sequence of Events

In the twelve days during which tests were conducted, from November 30th to December 20th, a total of 26 runs were made, 20 of which reached a maximum speed of more than 450 km/h. The maximum record speed was reached on the fourth day, during the eleventh run. Speed progression had been rather fast as the following figures show:

415.9 km/h on the first day
442.6 km/h on the second
473.7 km/h on the third

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The following mileage was run:

903 km above 400 km/h
601 km above 420 km/h
381 km above 440 km/h
238 km above 450 km/h

It should also be noted that several passing-by of trains could be made with train 325 on track No. 1 and train 308 on track No. 2, with a maximum relative speed of 777.7 km/h, and the actual distance of 4.2 m between track centerlines.

V.2 Train Performance

At a maximum power of 13,000 kW, the wheel traction force was still around 100 kN at 460 km/h, giving good traction performance for the 290 T train at those speeds.

Thermal conditions remained also good for the electrical equipment (traction motors, transformer . . .), and no difficulties were encountered due to the high power transmission required.

Aerodynamic drag, above 450 km/h, was giving an equivalent 8 km/h/km deceleration. All these results show that the whole TGV system, designed for high power transmission and high traction forces between wheel and rail, is largely satisfactory, even in its initial design, at much higher speeds than commercially practiced.

V.3 The First Technical Results

Considering the great amount of data which could be gathered during these tests, it is easy to understand that only part of them have been treated by now, and that only the very first results can be presented here.

1. *The dynamics of the vehicle on the track*

It should be noted, at first, that the track geometrical quality was very good. The mean absolute value of defects, as recorded by the Mauzin coach being the following:

longitudinal level	NL = 21 mm
transverse level	NT = 21 mm
alignment	D = 28 mm

By comparison, during the November 1988 tests on the South-East Line, geometrical quality could be brought to the following figures—but this was on a track section opened to commercial service, seven years after construction.

NL = 42 mm
NT = 37 mm
D = 54 mm

Such a high geometrical quality is not absolutely necessary as the margin to safe limits shows, but gives a very comfortable ride, which was underlined by all visitors at the test.

On the vehicle, the superior bogie stability which had been anticipated by theoretical models was confirmed by experience. From the wheel/rail forces recorded data and from track geometry records, correlations will be made to develop simulation methods. The overall figures which have been recorded stand as follows:

- maximum lateral force exerted by a driving axle: 48 kN, or 85% of the *PRUD'HOMME* limit
- maximum lateral force of a trailing axle: 35 kN or 70% of the *PRUD'HOMME* limit
- maximum truck lateral acceleration: 6.8 m/s^2
- maximum body lateral acceleration: 2.8 m/s^2.

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Concerning this last figure, it should be noted that the following cant deficiencies could be reached:

170 mm at 392 km/h in 6500 m radius
157 mm at 398 km/h in 6250 m
117 mm at 448 km/h in 10,000 m
99 mm at 482 km/h in 15,000 m

Also, the observed maximum lateral body acceleration occurred in a curve with some 165 mm cant deficiency, giving some 1.4 m/s^2 permanent lateral acceleration.

Comfort in the highest speed section was still very good above 420 km/h, with values of:

- 8 to 12 hours in the lateral direction,
- 15 to 16 hours in the vertical direction in a trailer coach.

2. *Dynamics of the pantograph and catenary*

Current collection was excellent up to 440 km/h on the 28 kN tensioned zone, remained good up to 470 km/h, then somewhat degraded. None of the arc was harmful to the current intensity transmission of 700 amp, which was necessary to reach the 482 km/h maximum speed. Uplift stayed at a 31 cm maximum at suspension, leaving a good margin to the 40 cm allowable limit.

This excellent behaviour of the pantograph-catenary system shows that improvements and better performances are still possible by means of rather small alterations of a system which had never been expected to pick up such intensities through so small a contact surface.

3. *Track measurements*

In the plain track, in 15,000 m curve radius with 98 mm cant deficiency, minimum vertical load measured (in discrete points) was 38 kN at 442 km/h (and not at the maximum speed) on the high rail; maximum measured lateral force on the high rail was 27 kN at 450 km/h.

In fact, as could be derived from the on-board measurements, the forces on the track on the whole test section were always well below any limit which would have produced a change of track geometry.

This good behaviour can also be observed from acceleration levels recorded on the sleepers, with peak values of 30 to 40 m/s^2, and in the ballast, with current peaks standing at 10 m/s^2, and only a few higher values up to 15 m/s^2. These values are of the same order of magnitude as those normally observed under more classical passenger trains at 200 km/h.

4. *Environmental measurements*

Assessment of the environmental effects of a passing TGV are still going on. At first glance, the following conclusions can be drawn:

- For noise emission, aerodynamic noise is mainly increasing above 360-400 km/h. Hence noise evolution with speed is building up following higher order power law from 400 to 480 km/h out of lateral noise protection walls
- For ground vibration emission the results are largely dependent on soil nature and transmission characteristics, as can be derived from the differences between measurements made along the South-East Line in 1988 and along the Atlantic Line in 1989. However, in all cases, ISO standard 2631 limits for night dwellings are complied with at distances from the track of 18 to 25 m, at speeds between 350 and 400 km/h, without any vibration deadening system in the track,
- For the wind effect of a passing train, the most important side-track effect arises from turbulences created when the tail of the train passes by. Slightly different values were obtained between the tests on the South-East and Atlantic Lines, the latter being more favourable.

A 100 N figure was measured as the aerodynamic drag on a cylinder representing a human body set at 2 m from the track when the nose of the train was passing by at 350 km/h. As the tail passes by, this figure is increased to 150 N at 2.4 m from the track, which is still sustainable by the staff standing along the track.

VI. CONCLUDING REMARKS

The complete series of tests which have been described here will bring a large amount of new data about the possible limits of the already existing TGV system, particularly in the following domains:

- suspension systems,
- truck stability and behaviour,
- behaviour of mechanical elements such as axles, axle-boxes, gears . . .
- pantograph and catenary,
- track superstructure behaviour

Being realized with a standard series trainset, they have also demonstrated the very large margin of the system, when actually in commercial service at 300 km/h. Moreover they enabled us to pick the 300-350 km/h domain out of pure scientific research investigation and speculation to set it as the next industrial development phase of the TGV system.

More testing will have to be done in the near future to cope with this objective, and it has already begun. On the 29th of January 1990, all the staff was back on trainset No. 325 to open track No. 2 of the same line section at speeds between 400 and 460 km/h. On this very first day of running on this track four turnouts were crossed in Vendome at a maximum speed of 462 km/h, giving another proof, of the very high potentialities of the classical wheel/rail TGV system.

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By: W. M. Worthington*, D. J. Barkley*, J. S. Stover*

Conrail maintains some 15,700 road miles of track in the Northeast and Midwest section of the United States. To facilitate the objective appraisal of our rail in track we purchased a dual rail Liteslice System in 1985. This system was installed in our Rail Analyzer Car along with a Rail Corrugation Detection System. This Rail Analyzer Car generally operates in tandem with our Track Geometry Car.

Since that time we have progressed the development of a post processing system that yields accurate objective information on vertical and horizontal head wear, thereby giving us the ability to rank rail segments on the basis of head area loss. This system allows us to minimize the size of our rail renewal program and insure we are replacing only "worse case" worn rail. The ranking system is used to provide us information on the worse worn rail on a given length of track. As a result, rail stretches can be ranked on the basis of wear severity.

Because of our confidence in this process, we have been using it to determine our curve patch requirements for the past three years.

Figure #1 is an example of a Rail Ranking Report for our River Line. The process has selected the most severely worn rail (Determinant Rail) upon which to make its ranking. Note that the accumulated mileage requested in this report totals 3.07 miles.

Maximum and average vertical wear as well as maximum and average horizontal wear (in 1/16 of an inch) is also available in case field verification is required. During a typical test, data frames are taken at 30 foot intervals at speeds of up to 60 mph. In Real Time as well as in the post processing stage, those data frames containing poor or insufficient data are rejected.

During the post processing stage curve data from the Track Geometry Car is utilized to select only those optical data frames occurring within curve limits. To insure that the Track Geometry and Rail Analyzer Cars operate in a synchronous manner, a "LAN" was developed that ties all the on-board systems of the two measurement cars together.

The point of radius of the web base fillet is used as the reference to establish the height of the rail in track. (Figure #2). Comparison to the ideal section yields the amount of rail head wear in 1/16ths of an inch for each data frame. Averaging of the head wear for those data frames throughout the curve yields the average vertical wear values shown in the ranking report. Examination of each data frame is then made to determine the maximum vertical wear shown on the report.

To determine the horizontal wear, the center line of each head profile is established by first translating the data points to the proper coordinate system and then investigating the point at which the XY coordinates (data points), making up the rail head crown change from a positive to a negative "x" value. At this point we project downwards 5/8ths of an inch and then horizontally in both directions to determine wear on the gauge and field faces of the rail. Averaging of those data frames within a curve yields the average gauge wear for that curve. Again, the data frames are reexamined to determine the maximum gauge value for each curve.

Following the translation of the data points; (X,Y) to the proper coordinate system "Simpson's Rule" is applied between the end points of the rail head profile to determine the area of that head. The end points must first be extended downward to compensate for the "black" area beneath the head where flash lamp light, in many cases, does not penetrate. This extension places the end points on the tangential projections of the top fillet. This calculated head area is then compared to the same area of the new rail for that track location. The percent head area loss for each data frame is determined and then compared to the table shown in Figure #3 for numerical rating purposes.

---

*Maintenance of Way Dept., Conrail

Figure 1

1990 Patch Rail Ranking
Philadelphia Division

				Curve			Determinant Rail						Opposite Rail						
									(1/16 in. wear)						(1/16 in. wear)				Actual
Line Name	Last Tested	Trk.	Test Dir.	MP Beg.	MP End	Hand Crv.	Rail & Rail Wt	Rank	Max VT	Avg VT	Max GA	Avg GA	Rail & Rail Wt	Rank	Max VT	Avg VT	Max GA	Avg GA	Track Miles
River Line	8-12-89	007	N	22.48	22.59	L	E 127	875	9.	8.	9.	8.	W 127	709	7.	6.	4.	2.	0.11
River Line	8-13-89	007	N	61.97	62.11	L	E 127	863	8.	7.	12.	10.	W 127	647	6.	5.	2.	1.	0.25
River Line	12-15-89	007	N	50.94	51.05	R	E 127	862	10.	9.	9.	7.	W 127	600	6.	5.	4.	3.	0.36
River Line	8-12-89	007	N	40.38	40.70	R	W 127	852	9.	8.	15.	12.	E 127	748	8.	7.	8.	5.	0.68
River Line	8-12-89	007	N	22.17	22.33	L	E 127	814	8.	7.	7.	5.	W 127	728	7.	7.	3.	1.	0.84
River Line	12-15-89	007	N	77.14	77.27	L	E 127	811	8.	8.	6.	4.	W 127	768	8.	7.	3.	2.	0.97
River Line	12-15-89	007	N	54.38	54.53	L	E 127	810	9.	8.	4.	3.	W 127	798	10.	7.	11.	5.	1.12
River Line	12-15-89	007	N	76.77	76.88	R	W 127	804	9.	8.	15.	10.	E 127	651	6.	6.	3.	1.	1.23
River Line	8-13-89	007	N	54.39	54.51	L	E 127	804	9.	7.	4.	3.	W 127	750	7.	7.	5.	3.	1.35
River Line	8-13-89	007	N	93.03	93.13	L	W 127	801	8.	7.	3.	2.	E 127	612	7.	6.	4.	1.	1.45
River Line	12-15-89	007	N	49.22	49.44	R	W 132	798	9.	8.	15.	12.	E 132	562	6.	5.	3.	1.	1.67
River Line	12-15-89	007	N	7.12	7.56	R	W 127	782	9.	7.	12.	10.	E 127	718	7.	7.	5.	4.	2.11
River Line	8-12-89	007	N	3.19	3.57	L	W 127	769	9.	8.	5.	3.	E 127	730	7.	6.	0.	0.	2.49
River Line	12-15-89	007	N	59.95	60.26	R	W 127	763	8.	7.	14.	9.	E 127	634	6.	5.	3.	1.	2.80
River Line	8-12-89	007	N	34.57	38.84	L	E 132	759	9.	7.	2.	1.	W 132	742	9.	8.	8.	3.	3.07

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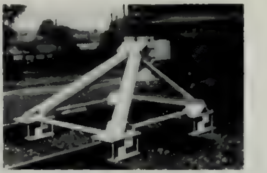

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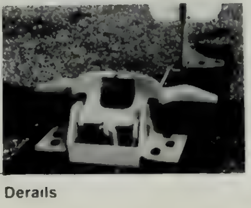

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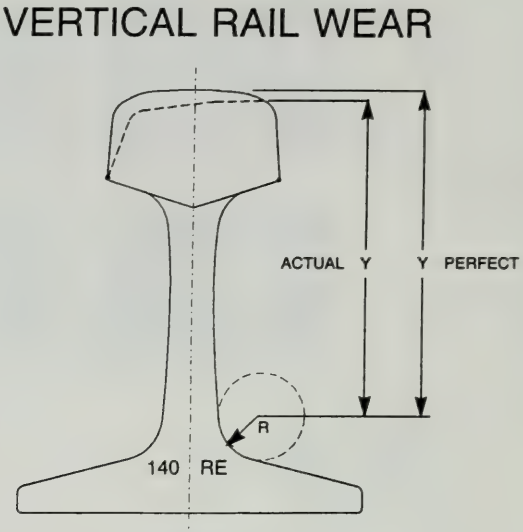

Figure 2

Percent Head Area Loss Table

Numerical Rating		152		140		132		127	
		High	Low	High	Low	High	Low	High	Low
0	New	0	0	0	0	0	0	0	0
200	New	5	5	5	5	5	5	5	5
400	MTR	23	16	23	19	20	17	17	13
600	BLR	34	23	37	30	33	25	30	22
800	S & Y	46	31	46	38	45	34	42	26
1000	SCRAP	75	75	75	75	75	75	75	75

Figure 3

Figure #2 reflects head area wear based on Conrail's wear standards using predicted wear patterns for the high and the low rails. As evidenced by the table, the weight of rail as well as its position in track (high or low) must be known. The percent head area loss of each data frame is fitted to the table using interpolation to determine its proper numerical rating. Subsequent averaging of the numerical rating for a curve yields a value which ranks it in relation to the other curves on a particular track segment. Those curves with the highest numerical ranking are flagged for possible inclusion in the rail renewal program. As a result of our success with this process, we now use it to determine our curve patch requirements by simply requesting the wear severity ranking of all rail in curved track between any two points on a line or branch. This ranking also provides a cumulative total for the curves involved thereby allowing us to tailor our requests to meet our budgeted rail allocations.

The system is presently being expanded to accommodate tangent track as well. Corrugation, rail defects, accumulated tonnage and grinding will eventually be added to allow us improved rail renewal predictive capabilities.

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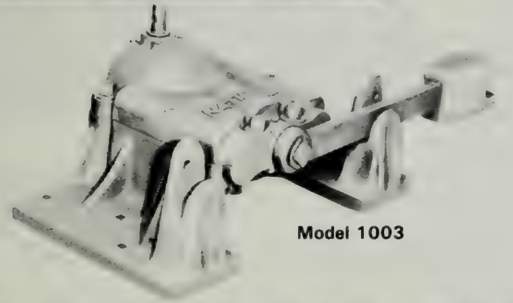

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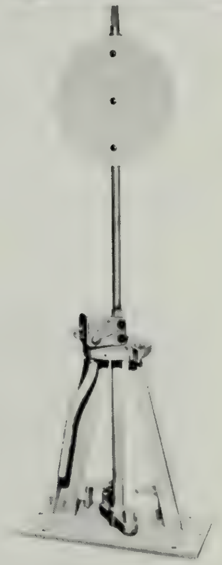

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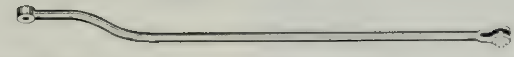

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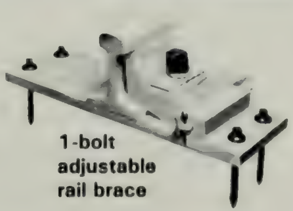

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SWING NOSE FROGS AND TANGENTIAL GEOMETRY TURNOUTS ON THE BURLINGTON NORTHERN RAILROAD

By: S. J. Sauer*

The first part of my presentation is on swing nose frogs used on the Burlington Northern. In the second part, I will discuss the use of tangential geometry turnouts on the Burlington Northern.

Swing Nose Frogs on the Burlington Northern:

The design and use of swing nose frogs has been around for a long time by other railroads throughout the world but mainly for lighter axle loads than what is used in the U.S. In 1982, the Mount Newman Railroad in Australia installed a swing nose frog on their iron ore mining railroad in Western Australia. Annual tonnage on this line is in the 50 MGT range with axle loadings around 70,000 lbs. After a trip by Burlington Northern to Australia, we decided to test one of these frogs in our 110 MGT line in Nebraska. This frog was installed in March of 1986 on wood ties. As of this date it has accumulated approximately 300 MGT. No welding repair has been performed yet. The only maintenance has been grinding. Normally, a railbound manganese frog would have been replaced by now. We expect to see 7 years of life from this frog on this line. However, it probably will not be scrapped at that point. We see a lot of rebuilding possibilities by replacing or rebuilding the worn parts. At this time, the only factor we see as determining the life of this type of frog is the amount of head wear on the rail and perhaps fatigue type defects in the rail.

I will not go into any details on the economic justification for this type of frog other than our studies indicate that if you can achieve twice the life of the railbound manganese frog with the swing nose frog, you can justify the purchase of swing nose frogs.

There are many variables that affect the justification, both hard and soft dollars:

- Quantity purchases of material
- Type of design of swing nose frog (I have seen prices ranging from $18,000 to $60,000)
- Life cycle costs of railbound manganese frog
- Switch machine-snow removal equipment costs
- Welders skills and track time availability to make repairs
- Train service interruptions caused by maintenance
- Exchange rates for purchases outside of the U.S.

Burlington Northern now has 38 swing nose frogs installed on our heavy tonnage lines, generally on lines of 70 MGT and above. The frogs are from several manufacturers with some design differences.

A quick primer on swing nose frog terminology:

"vee"—The point of the frog that moves from side to side. Similar in operation to a switch point.

"wing rail"—The portion of the frog that the "vee" rests against. Also similar to the stock rail of a switch.

"expansion joint"—A special joint at the heel of the frog that allows for free movement of the vee.

The following are some of the basic design types of swing nose frogs that I am aware of:

A. Swing nose frogs with welded vees of different construction:

1. Electro slag welded vee, no post heat treatment.

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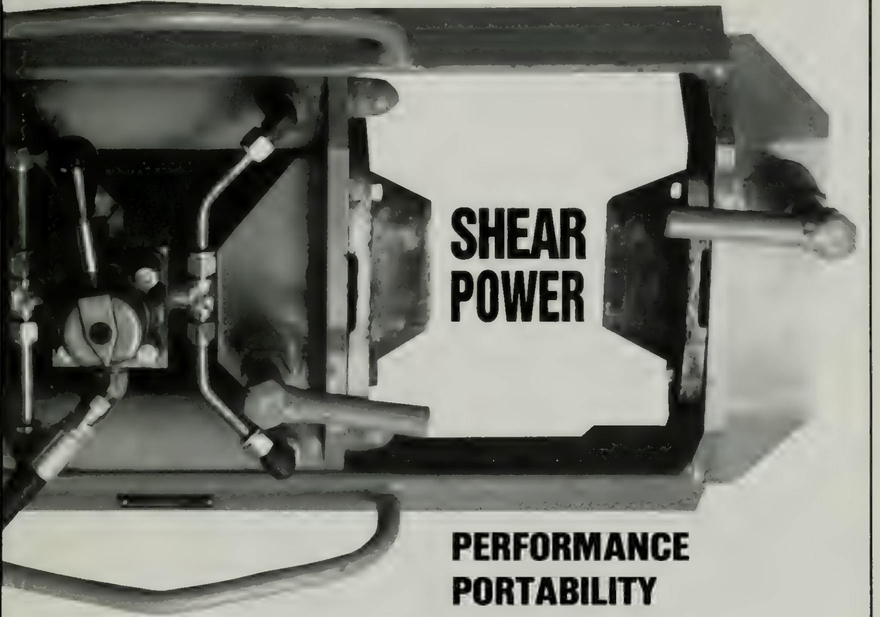
*Manager Standards, Burlington Northern Railroad



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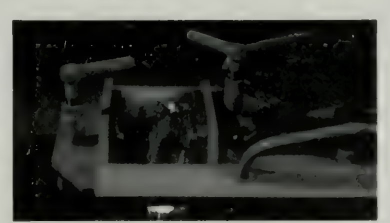

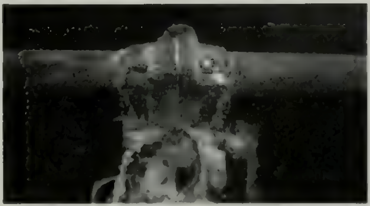

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2. TIG welded vee, post heat treated.
3. Lincoln Verti-Shield process with hard-facing overlay.

B. Swing nose frog with bolted vee constructions:

1. Vee is bolted together, wing rails or body of frog are made from cast manganese.
2. Vee is bolted together, wing rails are made from standard rail sections.

C. Swing nose frog with cast manganese vee:

1. Vee is cast manganese and so are wing rails or body of frog.

D. Swing nose frog with forged vee:

1. Front of vee is made from forging and then flash butt welded to rails. Rest of frog from standard rails.

Burlington Northern has installed all 3 welded vee types, and the bolted vee with standard rail for the wing rails. Most of these frogs have been installed on concrete switch ties.

Because of the problems we are experiencing with cast manganese, we decided to stay away from any of the cast manganese designs. Our experience so far is that wheel impacts are very low on the swing nose frog when compared to an open throat frog. Therefore, you do not need any special types of steels or alloys for handling the wheel impacts, such as austenitic materials like manganese. Any of the head hardened or fully heat treated rail steels perform very well, and can be weld repaired with standard procedures. Also, any improvement in rail steel will benefit the life of this frog.

Because we wanted a design with the fewest bolts, and hopefully reduced maintenance, we originally preferred the vees to be of a welded design. However, we have found that the manufacturers are having a difficult time in keeping the base of the vee flat. The welding process has caused vertical warpage or camber of the vee, and sometimes horizontal warpage. With this camber in the vee, the vee does not rest on all the plates. The remaining plates that are making contact with the base of the vee have increased contact stresses resulting in more friction, resulting in plate wear, and resulting in higher forces for the switch machine to move the vee back and forth. We have had train delays resulting from the switch machine not being able to move the vee to its closed position.

I am not saying it is impossible to weld a vee without the camber, because we have some, but I do recommend that railroads include in their purchasing specifications the amount of allowable horizontal and vertical camber.

Also I do not recommend using a riser rail for the false flange for the following reasons:

- It makes weld repair of the wing more difficult as it acts like a heat sink.
- The bolt that passes through the wing rail, riser rail, and brace is too long to allow for replacement without removing the frog from track.
- Extra expense

The best solution for the false flange appears to be machining of a groove in the wing rail.

Because of these problems, we decided to test a bolted vee type swing nose frog (Photo 1). Testing so far has shown that the bolted vee does not have any camber, vertical or horizontal, is resting on all the plates, and imposes a much lighter load on the switch machine.

In 1990, BN will install 9 of the bolted vee design swing nose frogs on #20 tangential geometry turnouts.

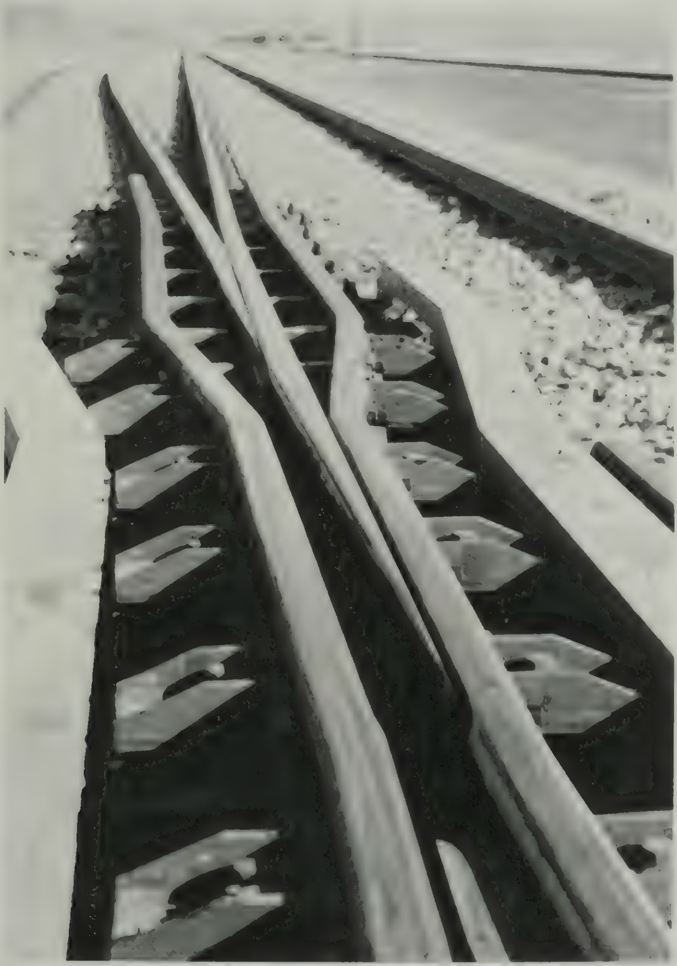

Photo 1 — Bolted vee type swing nose frog.

Tangential Geometry Turnouts on the Burlington Northern:

Burlington Northern Railroad has installed two #20 tangential geometry turnouts on Azobe ties with a bolted vee type swing nose frog. The first turnout was installed at Seward, Nebraska on a 110 MGT line and the second at Pacific Jct., Iowa on a 50 MGT line. Installations were in early November of 1989.

Burlington Northern current standard #20 turnout is the standard used by the former Great Northern, a predecessor railroad of the Burlington Northern. This slide (Photo 2) shows the geometry of the switch and the high switch entry angle.

Unfortunately, when the merger took place, Burlington Northern just adopted all the Great Northern standards without making a comparison between the other former railroad standards and choosing the best design. However, this probably saved a lot of fighting between the different railroad engineers.

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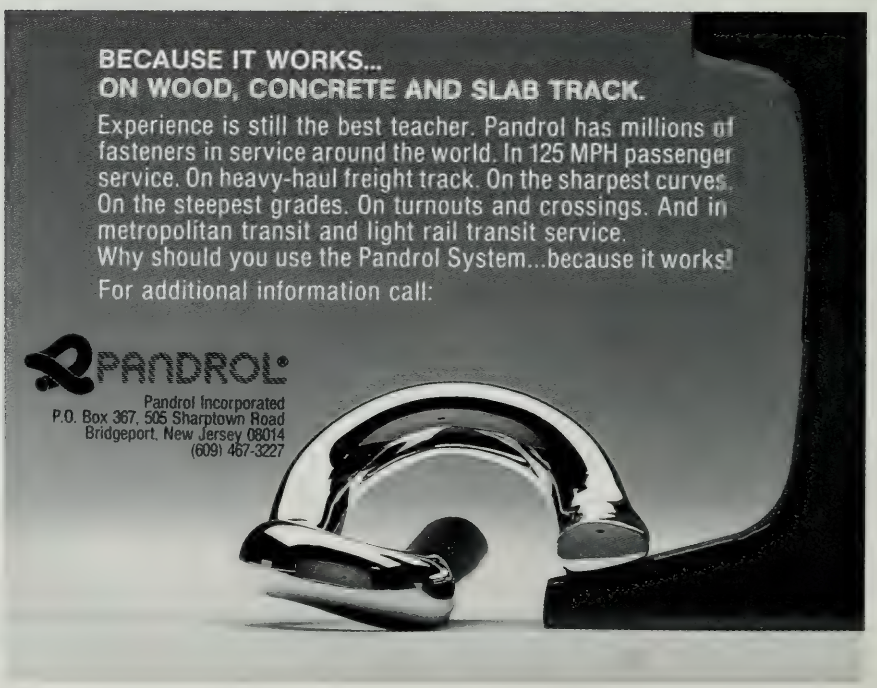

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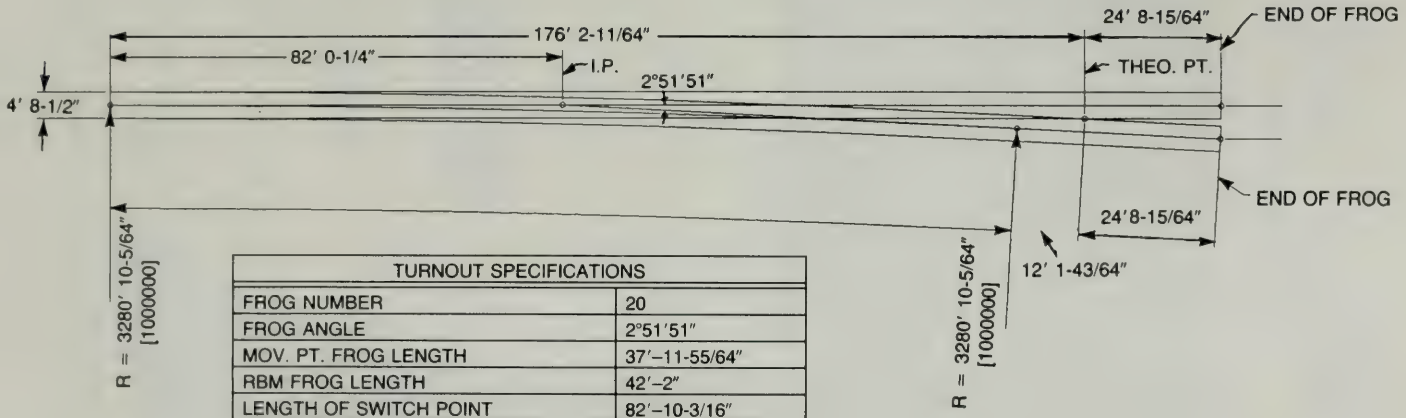
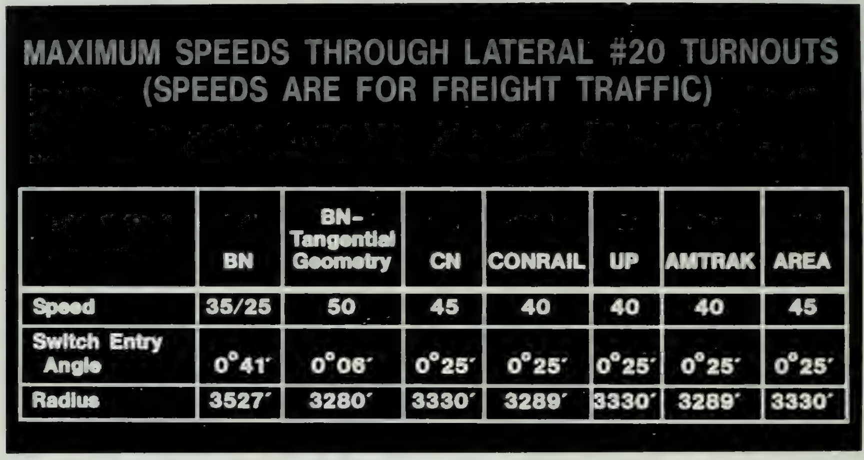


TURNOUT SPECIFICATIONS	
FROG NUMBER	20
FROG ANGLE	2°51′51″
MOV. PT. FROG LENGTH	37′–11-55/64″
RBM FROG LENGTH	42′–2″
LENGTH OF SWITCH POINT	82′–10-3/16″
ACTUAL LEAD FOR MOV. PT. FROG	163′–0-55/64″
ACTUAL LEAD FOR RBM FROG	173′–2″

Photo 2 — Standard #20 turnout on the Burlington Northern.

As a result, Burlington Northern's #20 turnout has the highest switch entry angle of any U.S. or Canadian railroad. This table (Photo 3) shows a comparison of some North American railroads with the Burlington Northern and AREA #20 turnout designs. The high switch entry angle results in high lateral forces at the beginning of the switch point causing two major problems—accelerated wear on the front portion of the switch point and problems keeping the switch in proper alignment.



MAXIMUM SPEEDS THROUGH LATERAL #20 TURNOUTS
(SPEEDS ARE FOR FREIGHT TRAFFIC)

	BN	BN-Tangential Geometry	CN	CONRAIL	UP	AMTRAK	AREA
Speed	35/25	50	45	40	40	40	45
Switch Entry Angle	0°41′	0°06′	0°25′	0°25′	0°25′	0°25′	0°25′
Radius	3527′	3280′	3330′	3289′	3330′	3289′	3330′

Photo 3 — Comparison of various #20 turnout designs.

Back in 1970 when the decision was made to use the Great Northern #20 turnout standard, average axle loads and annual tonnages were much less than today and so the switch entry angle was probably not perceived as a problem. Today, on our heavy tonnage divisions, the speed through the turnout side is restricted to 25 MPH in a turnout that has a radius that is good for 42 MPH based on 2 inches of underbalance. Burlington Northern will probably have more to gain by a change to tangential geometry than other North American railroads because of the high switch entry angle of the current standard Burlington Northern #20 turnout.

Burlington Northern has always used manganese tips on switch points to extend the switch point life and to provide a replaceable part for the portion that wears the fastest. With tangential geometry, the switch point will probably wear more evenly, eliminating the need for a manganese tip.

For Burlington Northern's #20 tangential geometry turnout, we decided to use a design that allowed for the maximum radius in a #20 turnout and still be able to use a straight frog. This radius is 1000 meters or 3280 feet, which is close to the radius used in our standard #20 turnout. At 3 inches of underbalance this turnout is rated at 50 MPH.

BN officials rode several trains through the turnout to determine the ride quality and to watch the turnout structure to see if there are any instability problems. The ride has been described as feeling almost like tangent track! We did not see any instability problems or excessive component wear in the turnouts.

Economic justification studies have shown that at some beginning and ending of double track locations, the payback from the increased speed is one year.

Burlington Northern's tangential geometry turnout uses some of the same design features and components incorporated in European turnouts. The following summarizes the advantages and disadvantages of the European designed tangential geometry turnout:

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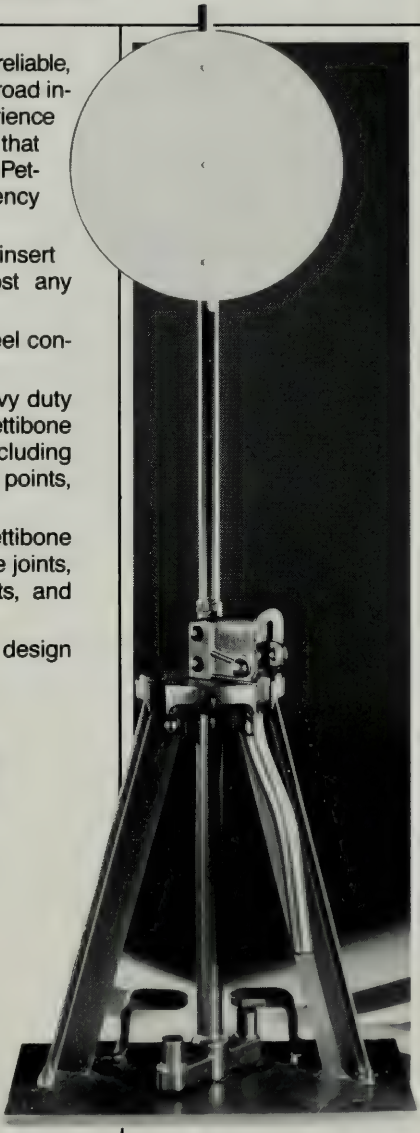

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Advantages:

Asymetrical Switch Points—

1. No bottom planing needed
2. Allows for fixation of stock rail on both sides
3. Eliminates need for braces
4. With long points, only 2 tie cribs filled with rods. Allows more ties to be tamped, fewer rods.
5. Only 5 holes needed on asymetrical point, all in base. On BN #20 T.O., there are 48 holes drilled in web.
6. Easier to manufacture, fewer holes, less planing operation

Tangential Geometry—

1. Eliminates large switch entry angle and lateral impacts
2. Manganese tip no longer needed
3. Switch alignment should be easier to maintain because of reduced impact.
4. Turnout in curves easier to design, construct, and maintain.

Fan Shaped Tie Layout—

1. Eliminates need for left and right hand material

Disadvantages:

Asymetrical Switch Points—

1. Heel end of point must be forged and then flash butt welded to standard rail section
2. Few suppliers of fully heat treated or head hardened asymetrical sections rolled yet.
3. Helper rods required

Tangential Geometry—

1. Switch points must be around 80′ long
2. Lead must be lengthened to achieve desired radius

Fan Shaped Tie Layout—

1. No major problems occured during installation due to the fan shape.

Tangential geometry allows the designer of the turnout to use standard equilibrium equations to design for a desired speed without having to reduce that speed because of a high switch entry angle. Previously, North American railroads used equilateral turnouts to achieve higher speeds through the turnout. With the tangential geometry, you can operate at a higher speed through the curved side of the turnout.

In 1990, Burlington Northern Railroad will install nine concrete tie turnouts of the same design as the first two wood tie turnouts. These will be located in our heavy tonnage lines in Nebraska and Wyoming.

Conclusion:

These turnouts are more expensive than any of our current standards. Because of the increased expense we will only be installing these turnouts at locations where we can expect a good rate of return on the investment.

Burlington Northern's lines with high tonnages and high train densities have caused us to consider other solutions to the current standard materials and turnout designs. The swing nose frog and tangential geometry turnouts will help us reduce turnout maintenance costs and provide improved train performance.

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BRIDGE UNDERWATER INSPECTION AND SCOUR

By: F. G. Alden Burrows, Ph.D., P.E.*

1. General

Catastrophic bridge failure is one of the most serious problems encountered in the transportation industry.

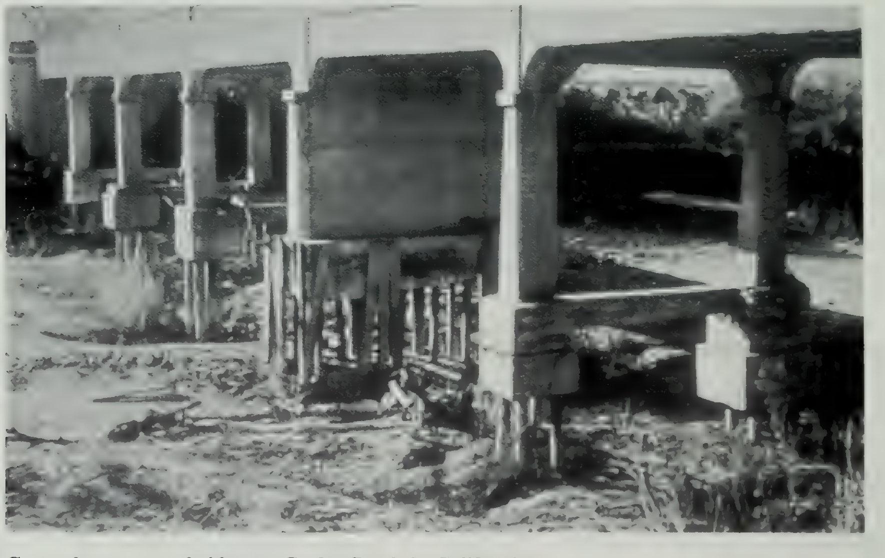
Bridge failure is mostly caused by severe floods, due to scouring of river channels.

Major floods can be caused by hurricanes, in particular the east coast; combinations of snow melt and spring rains in certain areas, and by rain alone in others.

There were seventeen roadway bridges, located in New York and New England, destroyed by snow melt and spring rains, in 1987. Over seventy roadway bridges were similarly destroyed in 1985, in Pennsylvania, Virginia, and West Virginia.

Hurricane Agnes, in a few days in June, 1972, killed several hundred people and caused several hundreds of millions of dollars damage. Many bridges were destroyed.

Four or five days of rain, in Northern California, at Christmas in 1964, caused loss of lives, severe damage, especially to railroads and roadways, and several hundred bridge failures.



General scour at a bridge on Cache Creek in California.

2. Total Scour

River channels are generally scoured by several different methods:

a. *General Scour.* This type of scour can occur in rivers with or without the presence of bridge crossings. Scour, which is called degredation, (building up the river bed is called aggredation) is caused by two different processes. The major one is caused by floods. As the water surface raises, the bed recedes, because of the increased river velocities and water depth. The magnitude can amount to several feet and can be over ten to twenty feet, depending on the magnitude of the flood and the gradation of the river bed material. Most often, as the flood recedes, the water depth

---

*Professor, Department of Environmental Resources Engineering, Humboldt State University

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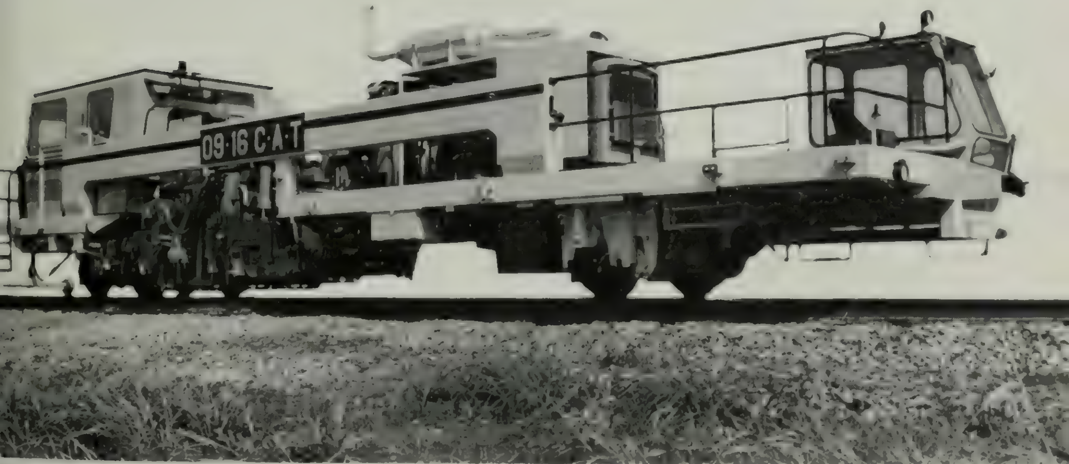

decreases and the river velocities slow down. The river is no longer capable of carrying the sediment and deposition results. The river, if in equilibrium, would have a very similar bed elevation as before the flood. It would appear that no scour, during the flood, had occurred.

The second kind of General Scour is usually caused by man made events. One way is the generally lowering of the river bottom because of mining of sand and gravel in the river. The result is a lowering of the river bottom, both upstream and downstream of the operation. The bed can be lowered for many feet and for many miles upstream and downstream. The river bottom in the tributaries are mostly also lowered. A similar result happens when river flow is added to a natural river due to the diversion of drainage ditches into a channel. Another effect is the raising of the river bed, due to, for example, removal of comparatively massive amounts of water for irrigation. The river surface generally also raises when the river bottom aggrades. The rise in water levels could impact the superstructure in floods, and could cause scour around the abutments. The main problem is that rivers are dynamic, often responding to changes in the river channel that the bridge designers never envisioned.

The Ead's Bridge, on the Mississippi River at St. Louis, was constructed in 1874. It was well known as one of the first major crossings of the Mississippi and was the first use of caissons in the U.S. for construction of the piers. At the time of construction, there was approximately thirty-five feet of water, then another thirty-five feet of sand to the limestone bedrock. The pier footings were constructed directly on the bedrock. The sand was excavated by use of the caissons. The piers were masonry, with each block expertly carved and placed. A magnificent effort. The piers were inspected one hundred years after construction, in 1974. There was seventy feet of water to the limestone bedrock. There was not a trace of sand. There has been sand mining in the area formany years which has helped to produce this effect. The cause is also because of dams that have been placed on the main stream and the tributaries during that time period, that intercepted the normal sediment flow in the river.

An extreme example of the amount of sediment moved during floods is the Eel River in Northern California. During the 1964 flood, it was estimated by USGS that 170,000,000 tons went out the mouth of the river, into the ocean, in three days.

b. *Local Scour.* Local scour is caused by local obstructions to stream flow, such as large boulders, bridge substructure units and timber debris. The scour is caused by fluid acceleration and by vortices in the flow. The fluid acceleration and vortices cause the bed material to be lifted up into the flow and to be moved downstream. The problem is increased when there is timber debris accumulated around river piers. The debris also causes downward flow which increases the scour. A bridge pier on Highway 101, at Scotia, California, dropped about five feet in elevation due to the accumulation of debris at the pier, several years ago. Two superstructure spans then collapsed into the river.

c. *Contraction Scour.* Many railroads and roadways cross flood plains. The cost of a bridge per foot of length is much more than the cost of the embankment. The embankments generally extend into the flood plane. Consequently bridge crossings usually restrict flood flows. This results in a higher river surface elevation upstream than downstream, and a local increase in stream velocities which results in bed scour and scour around the bridge abutments.

d. *Other Factors.* There are other factors that determine the amount of scour at bridge sites:

1. Type of material in the river beds and banks. Silts and sands are extremely susceptible to scour.
2. Migration of the river channel. There are both natural and man made causes to channel migration.
3. Bank slope stability.
4. Direction of flow, especially at flood stages.

Abutment scour.

5. Pier shape. Streamlined pier shapes can cause much less local scour than blocky shapes.
6. Depth and type of substructure foundations.
7. River dredging. Dredging, usually for river navigation improvement, sometimes undermines river pier foundations.

3. Other Problems with Substructure Units

Substructure problems that can cause catastrophic bridge failure are as follows:

a. *Masonry Piers and Abutments.* Many railroad bridges were constructed in the late 1800's and early 1900's. Most of these had masonry substructure units. River flow over many years has often deteriorated and removed the mortared joints between the stone. This leads to displacement and sometimes shattering of the stone, which can lead to substructure failure.

b. *Concrete Piers and Abutments.* Older concrete substructure units were usually constructed of mass unreinforced concrete. These units usually have structural problems, not only because of being older and lack of quality control during construction, but also because they are unable to resist tensile forces.

Newer substructure units are mostly constructed of reinforced concrete. The reinforcing steel, although sometimes causes rusting problems, especially in a salt water environment, resists tensile forces.

Most structures have been constructed with quality concrete since the later 1940's. Sometimes, however, one or two poor batches of concrete can cause structure problems, mostly because modern piers generally have relatively thin walls.

River flow causes wear in poor concrete. Other factors such as freeze-thaw cycles, ice flow abrasion, and chemical reaction also can cause concrete deterioration.

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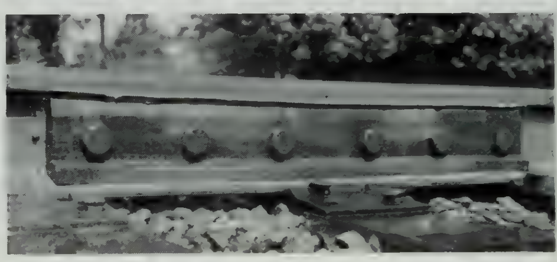

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Substructure units and piers in particular can also be severely damaged by river traffic on navigable rivers. Collisions with ships and river barges sometimes have completely collapsed river piers and caused superstructure units to fall into the river. Other times severe cracks and displacement have resulted from navigation damage.

Many older substructure units have been damaged by rail traffic. The older bridges were usually designed for large steam locomotives. The modern locomotive is generally lighter in weight than the old steamers, but now many trains are made up of heavy cars. The frequency of these heavy cars have caused problems.

4. Solutions

a. *Scour Damage.* New bridges can be designed to resist damage due to scour.

Existing bridges can be evaluated for the possibility of damage due to scour. The Federal Highway Administration has designed a manual entitled, "Hydraulic Engineering Circular No. 18, Scour at Bridges," which is scheduled for publication this year. The Highway Administration has asked the various states to identify "Scour Critical Bridges." The request stated in part: "Each state should evaluate the risk of its bridges being subjected to scour damage during floods on the order of a 100 to 500 year return period or more." Circular No. 18 lists guidelines for design of new and evaluation of existing bridges for scour. The evaluation of existing bridges includes underwater inspection.

b. *Structural Damage.* New substructure units can be designed for impact due to river collisions and also for live loads due to railroad traffic.

Existing units should have regular inspections above and below the water line. The inspections should include measurements of horizontal and vertical displacements and for significant cracking. Above water inspections generally can detect displacement. Underwater inspections can only detect large deflections. The substructure units should also be inspected shortly after collisions from river traffic, if possible.

Pier turbulence in a flood.

5. Underwater Inspections

Bridges with one substructure unit permanently in at least a five foot water depth should be periodically inspected underwater by an experienced engineer-diver. The time period between inspections should be from two to seven years depending on the condition at the last inspection and as to whether previously recommended repairs were done. The inspector should be experienced in bridge design and construction and in underwater inspections. Most inspections are qualitative judgements, and training and experience is needed to determine if damage is serious or is only cosmetic. The same qualifications are needed in order to recommend specific repairs. An inspector should also be able to recommend the time frame as to when repairs are needed. Usually repairs to substructure units are expensive and bridge owners would like to plan for the required funds.

Underwater inspections generally cannot be done during flood flows, but instead should be done when the river velocities are slow and the water relatively warm, so that the diver can concentrate on the bridge condition. Some rivers are somewhat clear so that inspections can be done visually, however, most are murky and inspections have to be done by touch. Underwater lights in these conditions usually reflect off the silt, like auto lights in a fog. This makes the inspections more difficult because many times there is timber debris, rope, wire and other objects around the substructure units. Underwater inspections can locate most exposed deficiencies, such as missing mortar and displaced and shattered stone in masonry piers, and spalls and cracks in concrete units. Hairline cracks usually can't be found, but they are generally structurally insignificant.

Divers can also determine if the foundations are undermined and also ascertain the erodibility of the stream bed material. The disadvantage of diving at slow velocities is that most scour holes are partially filled in.

Smaller bridges, sometimes with only one pier in the water, generally have more problems underwater than do large ones, because usually less engineering design time and money was spent on the projects. Many bridges, especially the older ones, don't have "as built drawings," and indeed many substructure units weren't built as originally designed. Detailed underwater inspections can correcct this problem on the exposed portions of the units. Underwater inspections can also determine the soundness of timber, concrete and steel piles. These piles are usually located in pile bents and below footings. Generally these piles have more damage in a salt water environment than those in freshwater. Timber piles can have dry rot and splits and in addition marine borer damage in salt water.

6. Below the Water Repairs

Scour or potential scour damage can be corrected by the following measures:

a. Providing rock or concrete filled sand bag blankets on the stream bed and at river banks around the substructure units.

b. Constructing channel improvements.

c. Strengthening the substructure units.

d. Constructing relief bridges on the flood plane, and or lengthening the main channel bridges.

Structural repairs to substructure units can be done by the following:

a. The most expensive, for most rivers, is driving steel sheet piling, dewatering, and rebuilding the unit.

b. Underwater repairs can be done, such as:
   1. Re-grouting mortared joints in masonry substructure units, by cement grout, epoxy grout, or material such as lead wool. Environmental controls have to be considered.

2. Cleaning the damaged areas, driving steel sheet piling and placing tremie concrete, without dewatering. Additional foundation piles may be required.
3. Cleaning the damaged areas, placing steel tunnel liner plate underwater and placing tremie concrete. Additional foundation piles may be required.

7. Conclusion

The underwater environment is extremely harsh. Many bridge piers have existed in that environment for over 100 years. On the other hand, some have collapsed in a short time. Substructure units should have regular underwater inspections, as part of a preventative maintenance program, in order to extend their useful lives. Underwater inspections are relatively inexpensive and past inspections, many times, have prevented bridge collapse, saving lives and many hundreds of thousands of dollars.

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NEW FRA REGULATIONS ON CAMP CARS

By: L. T. Cerny*

In 1989, there were two items on the FRA's agenda involving new regulations dealing with camp cars, which are also known as bunk cars or outfit cars.

The two camp car items involved first, the protective measures taken to avoid unexpected movement of, or impact into, occupied camp cars. The second involves regulations concerning the clean, safe, sanitary, and quiet aspects of living quarters in camp cars. This second item is still being considered by the FRA, but no date has yet been set for a hearing.

The first camp car matter, that of protecting the camp cars from unintended impact while occupied, was the subject of new regulations which went into effect January 1 and will thus cover all camp cars this year. Those of you familiar with the "Blue flag" regulations used by mechanical departments when they have people working on railroad cars will recognize that the new regulations are very similar, but use a different symbol. The symbol required as a warning signal is to use the exact words of the new regulation, "a white disk with the words 'occupied camp car' in black letters and an illuminated white signal at night." While the size and illumination levels are not made specific by the FRA, prudence would indicate an approximate size and illumination corresponding to blue flag apparatus presently used by your railroad.

Shown on Pg. 214 is the standard that the Burlington Northern has adopted for their Occupied Camp Car sign, with a detailed view of the sign itself. They use a flashing lamp for illumination at night.

The rule indicates that such "warning signals," indicating the presence of occupied camp cars, shall be displayed by a designated occupant of the camp cars or that person's immediate supervisor as soon as such cars are placed on the track.

After the cars are placed, each manually operating switch providing access to the track on which the camp cars are located must be lined against movement to that track and locked and spiked; or, if the speed on the track is restricted to no more than five miles per hour, a derail can be used instead of a spiked switch if the derail is at least 50 feet from the end of the camp cars. If speeds are over 5 mph, the derail can still be substituted for a spiked and locked switch if it is 150 ft. or more from the camp cars. The "warning signal" must be placed at the switch or derail as appropriate.

The rule would require a person to be present to remove the sign or light (and unspike any manual switch involved) prior to an occupied car covered by this rule being picked up by a train; or, after such a car was set out by a train, to place the sign or light and spike any manual switch. Where no manual switch is involved, other procedures are required by the rule.

The FRA has chosen to place no restriction on the speed of trains passing occupied camp cars on an adjacent track and also to place no speed restriction on the movement of occupied camp cars, but it does prohibit such cars from being humped unless coupled to a locomotive.

While this new FRA Camp Car "Blue Flag" rule excludes cars in wreck trains and cars while in any train, it would appear to apply, despite AAR-BMWE objection, to office cars, track geometry cars, and rail detector cars, if they are parked while providing overnight housing for railroad employees. It should be noted that the entire regulation specifically applies only to railroad employees, so that contractors' people or others housed in on-track equipment would not be covered.

Some railroads occasionally build temporary tracks for camp cars where a nearby track is lined over to the temporary track and then lined backed after the cars are placed, thus creating a situation where no turnout exists to the track on which the camp cars are placed. While the new regulations do not specifically cover this situation, common sense would indicate that no sign or light would be necessary, since there is no path for another train to contact the camp cars.

If the camp cars are placed on a track whose access is provided by a remotely controlled turnout, such as in an interlocking or in CTC territory, a number of additional requirements exist, and are listed in paragraph 218.77 of the regulations. These involve locking devices and certain record keeping.

If any problems arise regarding the new FRA camp car protection regulations, please feel free to call AREA headquarters.

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*Executive Director, AREA and Engineering Division, Association of American Railroads

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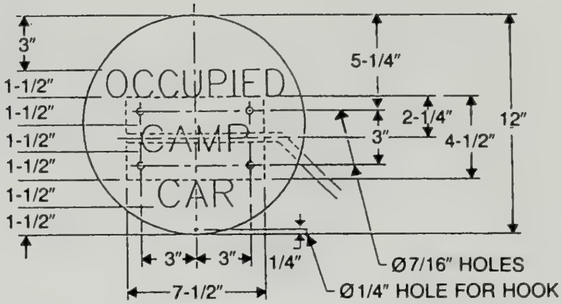
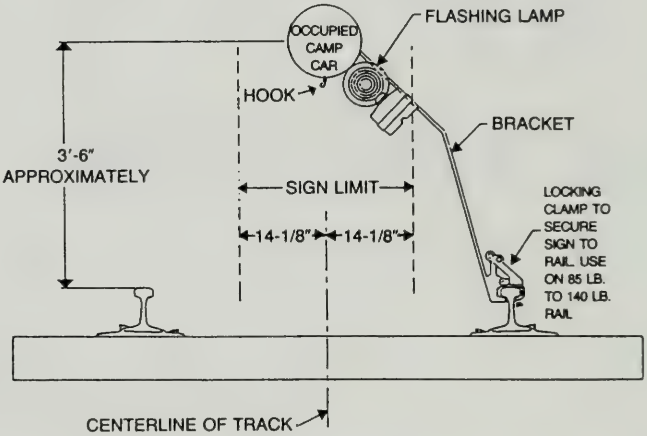


MOUNTING PLATE



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2. Facing: White engineer grade, enclosed lens, wide angle reflective sheeting, applied per manufacturer's instructions. The sheeting shall comply with the latest edition of Federal Specification #L-S00C.
3. Letters: Black silk screen or Scotchcal and conform to the Federal Highway Administration standard alphabet, series "D" 1966 edition.
4. Sign Support: Bracket to hold sign within middle one-half of track gage.
5. Sign manufacturer to supply an Empco-Lite Model 100-A flashing white light or equivalent, 12 volt batteries, and photo cell.
6. Sign manufacturer to furnish details of locking rail clamp and sign support for approval prior to manufacture.
7. Pyramid bolt used to secure lamp to bracket.

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COMMITTEE 16 — ECONOMICS OF PLANT, EQUIPMENT & OPERATIONS

Chairman: C. Bach

Report of Subcommittee D5-1-87

Subcommittee Chairman: R. W. McKnight

FACTORS TO BE CONSIDERED IN EVALUATING ADVANCED TRAIN CONTROL SYSTEMS

The Advanced Train Control System (ATCS) project, a joint venture of the Railway Association of Canada (RAC) and the Association of American Railroads (AAR) has reached the point where several railroads have begun to test various components of ATCS and subsystems. Much testing has been conducted by sending messages on the 890-935 MHz data radio between moving locomotives and wayside base stations. For ATCS applications in the United States, six frequency pairs in the 890-935 MHz range have been assigned by the Federal Communications Commission to the AAR, who is the radio frequency coordinator for the railroad industry. The Canadian government is also considering setting aside these frequency pairs for ATCS usage by Canadian railroads.

The RAC-AAR joint venture is nearing completion as far as specification writing is concerned by its consultants ARINC Research, Transportation & Distribution Associates and Lapp-Hancock & Associates. Guidelines for testing components and methods for system safety analysis are being developed.

One major railroad has announced plans to install a level 20/30 ATCS system on 180 miles of mainline over the next two years. Other railroads will probably follow.

Because ATCS involves a new type of train control, a study of its impact on operations is very important. Several present conditions and the operational status of the territory being considered for ATCS application should be studied. Additionally, study should be given as to how ATCS will affect operations and finally the economics and benefits determined. There are costs involved in changing over to operation under ATCS, but there are also some benefits that provide monetary and time savings.

Major factors to be considered in an ATCS study include:

- Geography
- Present control systems in operation
- Present and expected future traffic
- Number and estimated remaining lives of locomotives to be ATCS equipped
- Present communications
- Labor agreements
- Other operational considerations
- Training and retraining
- Maintenance

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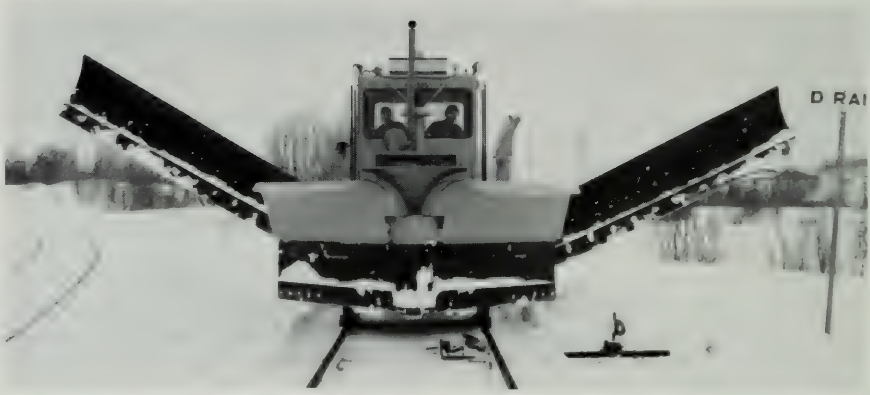

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More details concerning costs of these factors are:

Geography

Territory—Terminals, mileage.
  Grades, curves, bridges, tunnels, cuts, etc.
  Single, multiple track.
  Sidings.
  Crossovers.
  Urban, rural.
  Industries.

Present control systems

Signaling—Automatic block signals.
  Traffic control or centralized traffic control.
  Manual block.
Direct traffic control (radio manual block).
Timetables and train orders.
Other.
Maximum speeds permitted for each type of control system.

Present traffic

Number of trains per day by type and direction.
  Passenger, TOFC, stack, manifest, unit coal, unit grain, other unit trains, Road-Railer, locals, industrial assignments, transfer runs, etc.
Segment terminals of each train for territory under study.
Maximum permissible speeds by type of train.
Timetable speed restrictions for each type of train and mile post location.
Schedule or desired running time between terminals for each train.
Nominal tonnage handled by each train.
Horsepower per ton required to achieve desired schedule by type of train and by direction.
Number of opposing train meets and/or faster trains passing slower trains.
Flexibility of operating certain trains to minimize meets and passes.

Present communications

Telephones at wayside locations.
Two-way voice radio, end-to-end and train-to-wayside voice radio.
Data radio, one-way or two-way train to wayside.
Train-to-wayside coverage: continuous or intermittent.
Microwave, open-wire lines, cable or fiber optic cable to connect wayside locations with a central office.

Labor agreements

Locomotive engineers.
Conductors and trainmen.
Interlocking or tower operators.
Signalmen.
Communications technicians.
Dispatchers.
Clerical and/or computer operators/programmers.
Shop crafts.

Other operational considerations

Hot bearing detectors and other wayside detectors.
Interlockings and crossings with other railroads at grade.
Rail-highway grade crossings with or without warning systems.
Yards and terminals

Training and retraining

Train crews and dispatchers for operation under ATCS.
Maintenance personnel for on-board and wayside equipment.
Programmers and operators for computers.

Maintenance

ATCS is heavily dependent on computers and communications, especially data radio. ATCS will probably have more sophisticated equipment to maintain. There may be a need for more skilled maintenance personnel with ATCS compared to present control systems.

Programming and software development will be important. There is a vital and great need for documentation of software programs.

As for economics, each project will have to be studied and include at least the following:

- Costs for equipment
- Costs for programming and software development
- Costs for training and retraining
- Costs for installation
- Costs of maintenance
- Costs for overtime and recurring costs.

Any cost directly or indirectly resulting from the installation of ATCS should be included.

Indications are that ATCS will create a large amount of data and its resultant data flow. A key element to be considered is that all of the data generated must be useful in making decisions to make the railway operate more efficiently, safely and more effectively to meet customers' needs. Handling data efficiently so as not to inundate people with information and/or paper will be vital to the successful operation of ATCS.

Economies and savings

Implementation of ATCS will produce savings in operating costs in several areas, as well as improve service to customers.

Some of the savings should come from:

- Reduced fuel consumption due to pacing of locomotive performance with optimum meet and pass conditions pertaining. In better meet and pass planning and the on-board computer monitoring the locomotive condition and operating characteristics, trains will not run nor accelerate to high speed (high fuel consumption) and then wait at passing sidings for a considerable time to make a meet, or race to get to a yard that is temporarily plugged.
- Reduction in clerical costs at yard offices through remote data entry by the conductor regarding set-offs and pick-ups of cars. Thus, more prompt handling of work orders will improve service to customers. Also, an advantage of the ATCS work order system is the receipt of printed orders by the conductor eliminating any misunderstanding of verbal orders. Additionally, data radio transmission of work orders reduces usage of the voice radio.
- ATCS will foster centralized dispatching with savings resulting from a reduction in the number of dispatchers required. Such centralization will bring all movement information into one location where it can be expeditiously handled. Also, it will be found to be economical to centralize motive power dispatching at the ATCS dispatch center. This can make decision making more efficient at the railroad's operational headquarters.

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It's as true today as it was over one hundred years ago: there's nothing like wood for crossties.

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- Economies can be realized with the ATCS communications providing performance monitoring of not only motive power but wayside devices such as defect detectors, highway grade crossing warning devices, etc. Remote diagnostics can be performed, thus only requiring maintenance personnel to visit field wayside devices to make repairs or replacements.
- Economies in ATCS implementation will be realized through its standardization and modular components. Designed on the building block approach, ATCS can be upgraded from lower levels to higher levels without replacing equipment.
- Although not to be quantified until ATCS is in service, the concept of better and safer control of train operations should produce efficiencies in the line-haul and terminal operations because of the improved controls.
- Condition monitoring of motive power should produce economies in locomotive maintenance and servicing. As problems develop on line of road, motive power condition and diagnostics can be detected and reported promptly from the moving train to central dispatch or to motive power maintenance facilities. A real benefit is the detection of problems that otherwise are not detected until failure. Big savings are obtained in correcting before failure. Also, these maintenance facilities will be able to better plan their work, as well as handle incoming locomotive repairs promptly. With ATCS prompt reporting of failures, corrections may be made at the next terminal rather than taking the unit out of service. In some instances with the data radio capability of transmitting information to the locomotive display unit, minor corrections may be made enroute.
- The same increased efficiencies can be obtained in yards with prompt reporting by train crews of cars set off or picked up enroute. Yards are then able to plan their work better and do it more efficiently.

There are savings in the present cost of operations and improved efficiencies that can result from implementation of ATCS.

It should be mentioned that some railroads have already obtained some of the economies and benefits of ATCS by use of other systems and operational techniques. For example, at least three major railroads have had centralized dispatching in service for a year or more. Many roads have reduced yard clerical staffs through the use of computers and paperwork automation.

As for safety, ATCS operation must be compared with a railroad's present method of operation in the territory on which ATCS would be installed. Here again, in some territories some railroads have obtained a high degree of savings with other methods of train operation.

Thus, the benefits of ATCS will depend on each railroad's operating practices and management philosophy.

ATCS provides a new way to operate a railroad. More importantly, savings from lower costs and a more efficient and safer operation are available.

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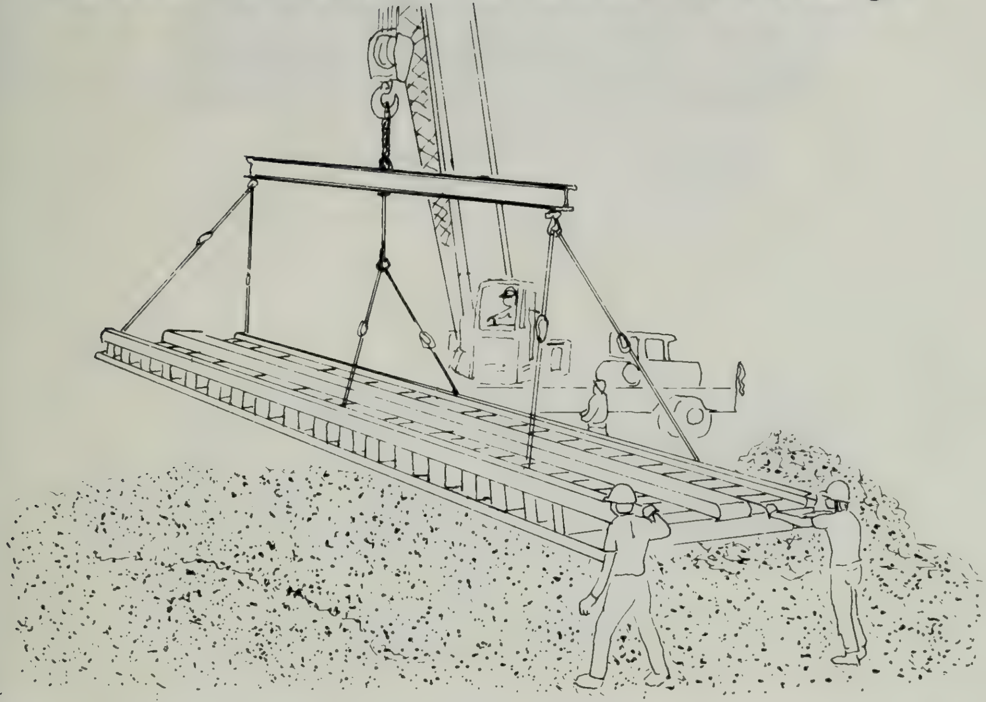

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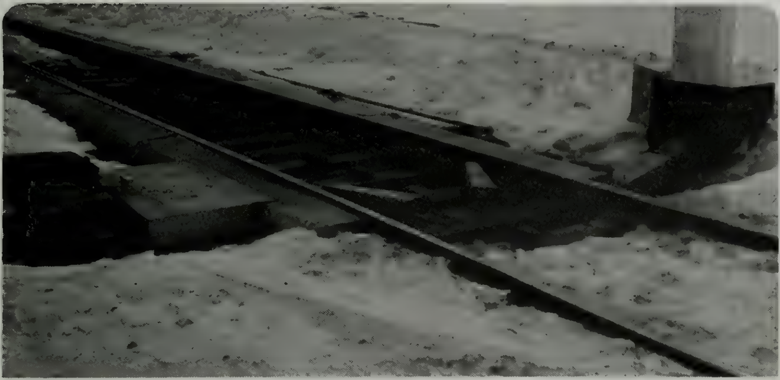

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Cover photo: Canadian National mainline, 120 miles west of Sioux Lookout, Ontario

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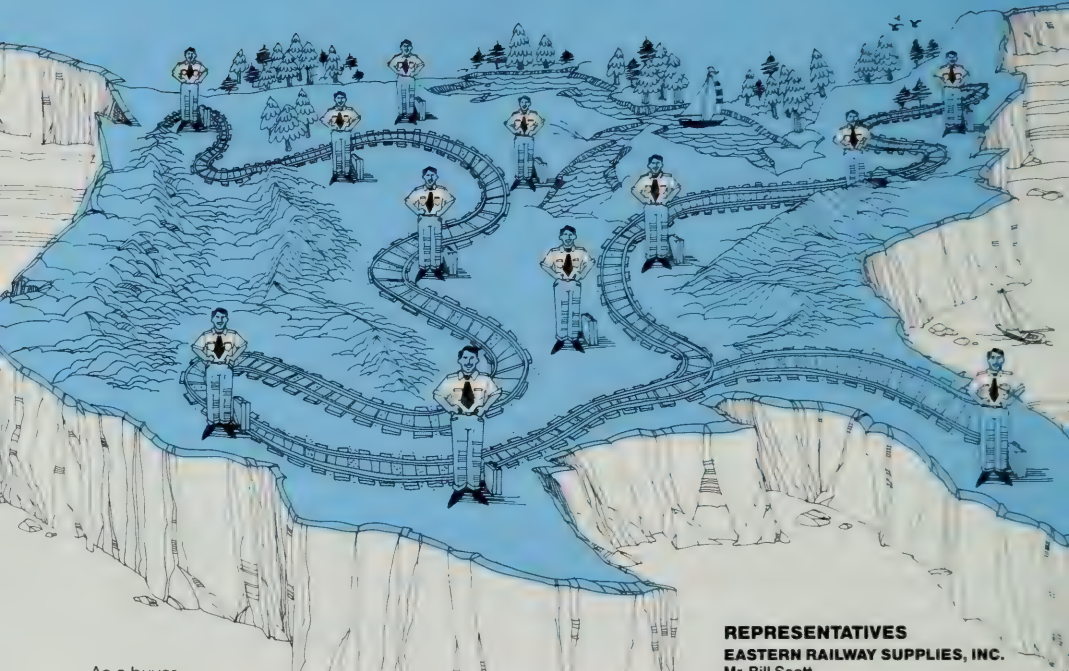

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Across the Canadian Shield

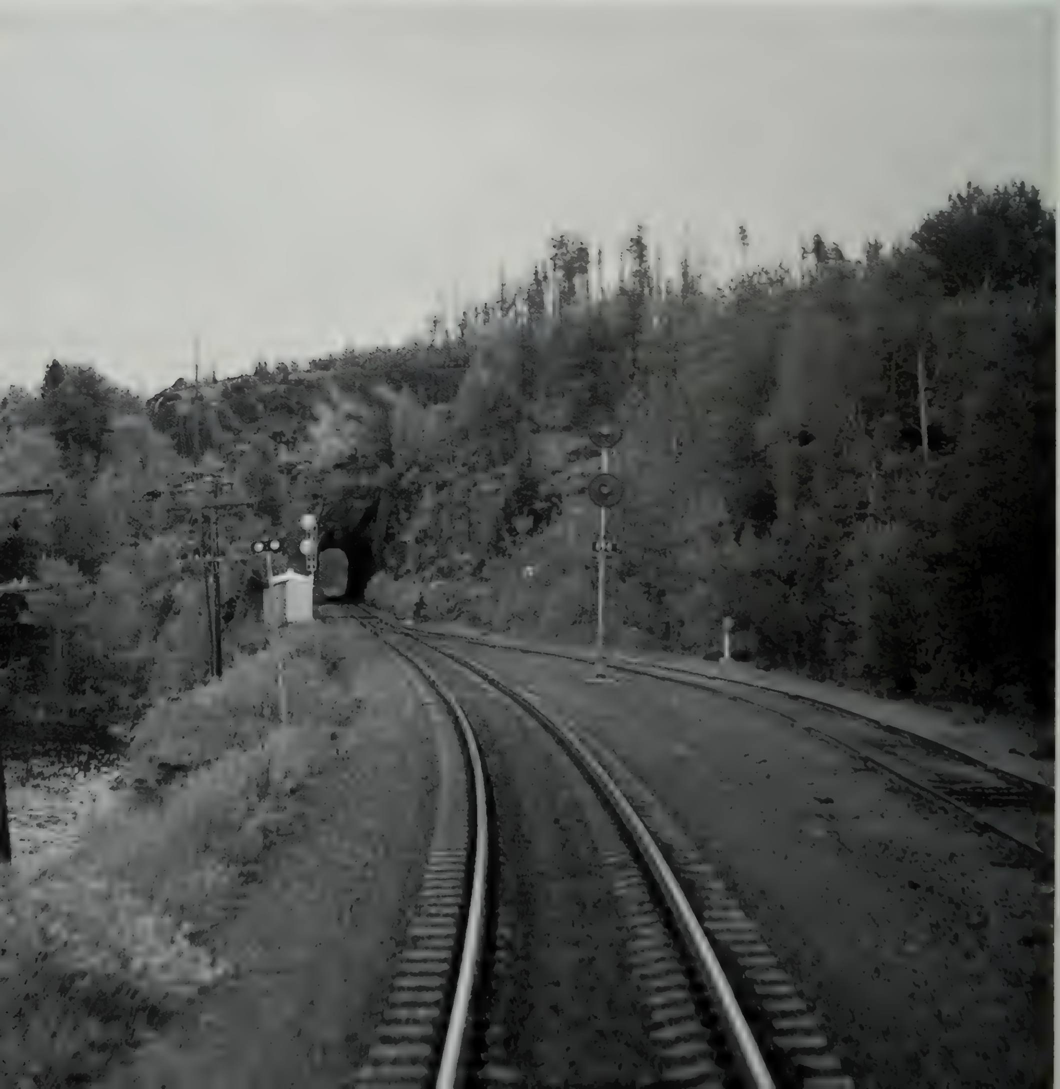
North of Lakes Superior and Huron in Ontario, and continuing east across Quebec into Labrador, lies the Canadian Shield. This land consists of a continuing jumbled maze of low bedrock ridges, usually covered by a thin layer of soil supporting coniferous and aspen forests, except where the weather has left the rock bare. These ridges are interspersed with lakes and swampy meadows connected by streams with rocky rapids. This area is largely wilderness, much of it without roads. While there are two railroad lines to the shores of Hudson Bay, there are still no highways reaching that body of water through Ontario. In addition, going from Eastern Canada to the Canadian

Above photo shows Canadian National mainline at wood to concrete tie transition point about 90 miles west of Sioux Lookout, Ontario. Photo below shows scene further west.

prairies and Rockies, all transcontinental highway traffic in Canada has only one alternative west of Thunder Bay, Ontario, and this alternative is a single two-lane highway, even though there are two railway lines in Canada from Atlantic tidewater to the Pacific.

The Canadian National runs through about 1000 continuous miles of this lake-and-rock-ridge terrain on its way east from the Canadian prairies at Winnipeg to tidewater on the St. Lawrence River. The disordered geographic arrangements of the lakes and ridges most often preclude an easy route through this type of terrain, making it necessary to cut through ridges and fill across lakes. The pictures in this article and on the cover were taken between 90 and 120 miles west of Sioux Lookout, Ontario on the route from there to Winnipeg.

While railroad tunnels are not common on the Canadian Shield, some exist, such as this one on the Canadian National about 90 miles west of Sioux Lookout.



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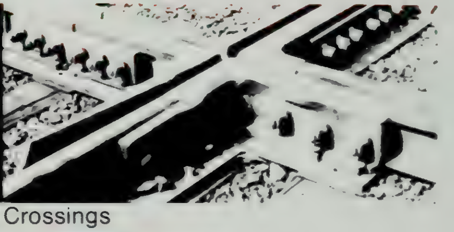

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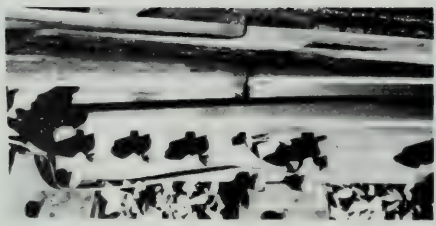

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MARCH 26, 27 and 28, 1990

REPAIRS TO CRACKS ON WELDED GIRDERS ON BURLINGTON NORTHERN'S LATAH CREEK BRIDGE

By: K. E. Bruestle*

On September 21st of last year, I took a day of vacation to catch up on some deferred maintenance work at home, and was accomplishing most of my objectives, when I got a phone call from my office at about 3:00 in the afternoon. I was informed that our Latah Creek Bridge had developed a major crack, and that traffic had been stopped across the structure. It was suggested that I put away my pruning shears and paint brush. I would be needing my hardhat where I was going. Travel arrangements were being made for me, and I was to meet our design consultant, HNTB, at the Kansas City airport for a flight to Spokane that afternoon. For those of you who are unfamiliar with the Spokane area it is at the Eastern edge of the State of Washington and, unlike Seattle, has a semiarid climate.

The Latah Creek Bridge is an extremely important structure to Burlington Northern, in that it handles about 80 million gross tons annually, with up to 50 trains per day. As can be seen in Photo 1, we have two mainline tracks converging at or near Spokane from either direction. Additionally, it is an AMTRAK route across our property. Convenient detour routes are not readily available.

Photo 1. Latah Creek Bridge near Spokane

This crossing of Latah Creek is at a very scenic, but challenging location. Our bridge is a high level one, about 200′ above the valley floor. The track is oriented east-west here, and runs parallel to Interstate 90. The bridge (Photo 2) consists of a 2100′ single-track tangent portion on the east end, with a wye on the west end. The total bridge length is approximately 3900′. The spans were designed as continuous steel box girders with a composite concrete deck. A cross section of the bridge in Photo 3 shows the steel portion and the crack location in the lower left quadrant. The steel is uncoated A588. This is, indeed, a beautiful structure . . . and that is not just *my* opinion, as an AISC award attests.

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*Director Design, Burlington Northern Railroad

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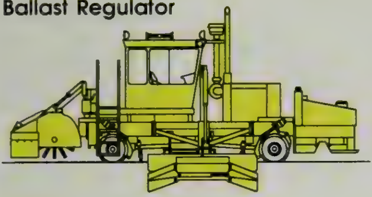

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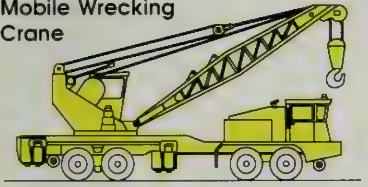

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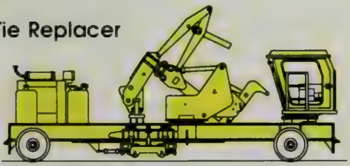

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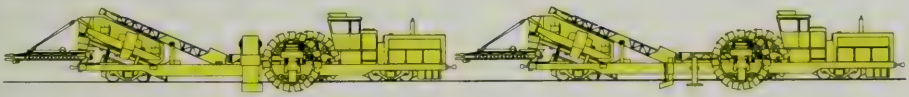

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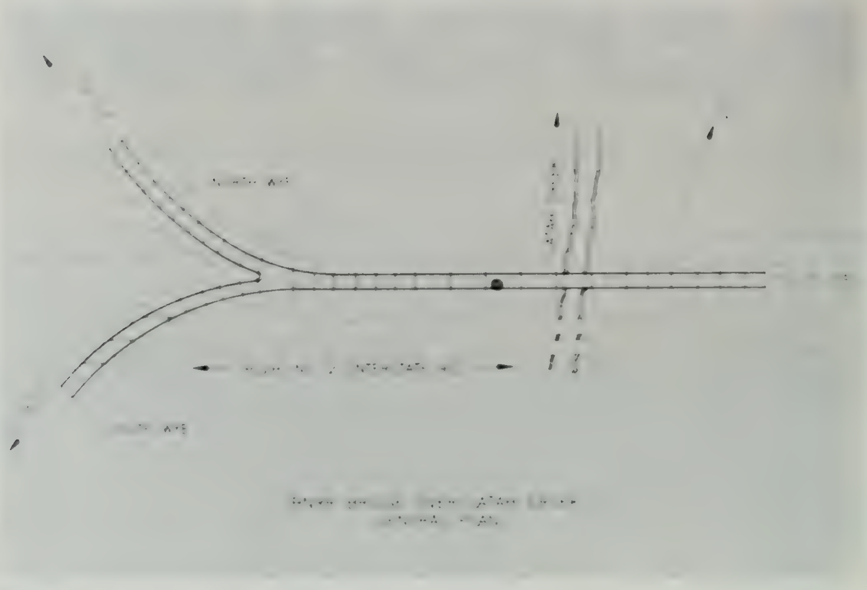

Photo 2. Plan of Latah Creek Bridge.

Our Latah Creek Bridge was built in 1971 and 1972 because of needs created by the newly-formed Burlington Northern, and some intense pressures being applied by the City of Spokane. The city was growing and could no longer tolerate our tying up street traffic downtown. In addition, Spokane was to host the 1974 World's Fair, and we had agreed to be off Havermale Island in time for construction on the island to be completed for this event.

We have experienced a history of weld problems with this structure, and have discovered minor cracking previously, through our inspection program, but never anything of this magnitude. This crack, too, was discovered during a routine inspection. Our bridge inspector was making his regular quarterly inspection, when he saw daylight shining through the web and flange plates. He realized that this was a potentially hazardous condition and stopped all train traffic immediately. But, it is not known how many trains may have crossed the span in that condition.

Calculations performed by our consultant subsequent to the incident indicate that even with this crack in the plates, we were in no danger of a catastrophic collapse. The span could have performed safely as a hinged cantilever under our heaviest trains. The compression flange stresses would have been about one half of the buckling strength of the section.

From the description of the crack in the steel, which had been conveyed to us by phone, we had a pretty good idea that a bolted sandwich-splice would probably be a proper repair procedure, at least as a temporary measure, so that traffic could be safely restored. So a tentative splice design was formulated at about 30,000′ above sea level while en-route on the plane. We arrived at Spokane at about 10:00 o'clock that night and proceeded directly to the site where we joined the Region Bridge Engineer and the local B&B Supervisor. They informed us that the crack was in span 10 from the east, or Spokane end. The only access to the crack location inside of the box is through inspection access trap doors at the abutments. The quarter mile trip from the access door, through the box, to the crack location is a particularly difficult one due to the interior bracing.

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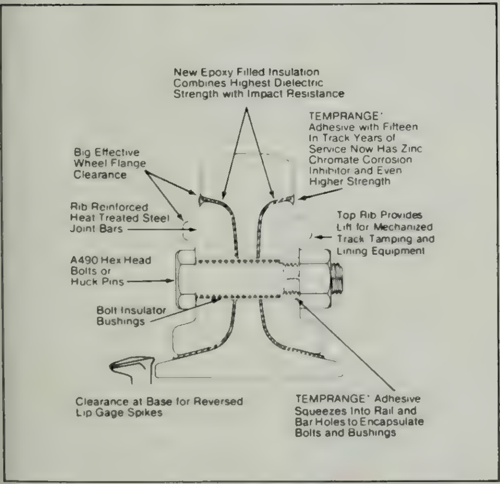
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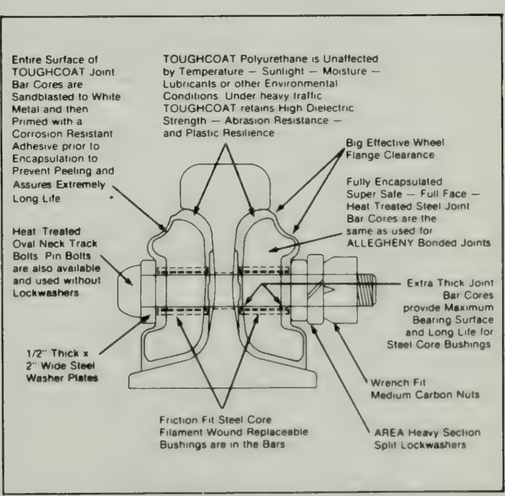
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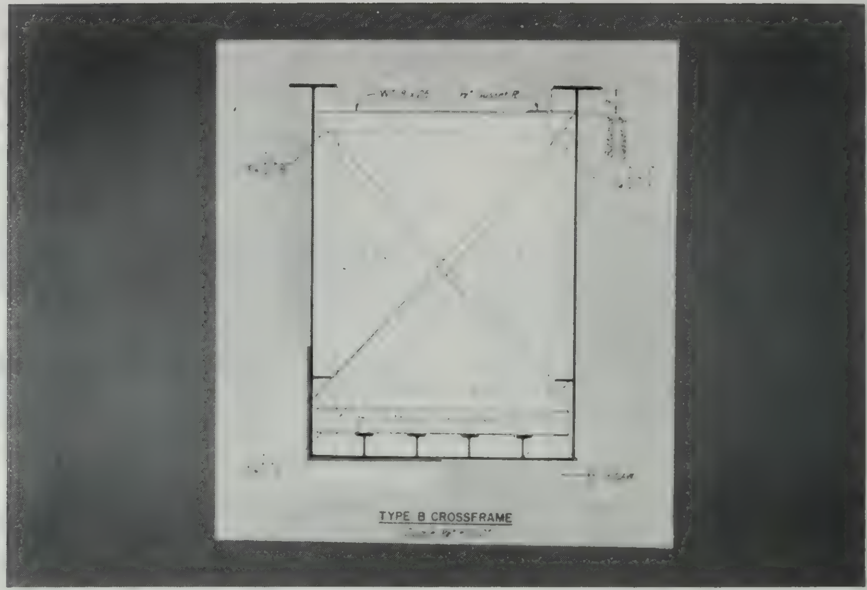

Photo 3. Cross section of Latah Creek span.

Everything that would be required to effect the repairs on the inside would have to traverse this route. Drills, torque wrenches, splice plates, bolts and nuts, food and water. Everything. At the piers, the challenge was even greater, where our B&B personnel had to squeeze through small openings in the diaphragms. The opening measures approximately 2′ by 3′.

When we finally arrived at the crack location, we were greeted by this sight. (Photo 4) The crack had apparently started in the flange-to-web weld and had propagated about 4′ into both the flange and web plates. BN forces had located the crack ends with a die penetrant and had drilled those locations to relieve the stresses and thus stop the cracks from growing. The crack had opened about an eighth of an inch and yet, when the UPI picked up the story, " The 8-foot crack on the underside of the trestle was . . . wide enough to stick a hand through." Had we been so fortunate, we would not have had to lug all the equipment and material that quarter mile through the box girder. We could have just passed it in through the crack.

At this point we fine tuned the splice design we had done on the plane and prepared a set of shop drawings for our fabricator, who had agreed to stay open all night to address our needs. We had a stroke of good luck here, in that the shop had an ample supply of high strength steel plates in any thicknesses we might need. It soon became apparent that the critical path on this repair work was going to be the drilling of the holes in the box girder web and flange. Three hundred holes would be required — 180 through the flange, and 120 through the web. If we waited for the fabrication of the splice plates to be completed before we started field drilling, we would have wasted several valuable hours doing essentially nothing. It was decided to make plywood and cardboard templates to match the hole patterns in the splice plates and begin drilling holes in the box girder web and flange plates before the splice plates arrived. The cardboard templates would be used on the inside of the box since they were easier to haul in that arduous quarter mile, and the plywood ones would be used from the underside.

We knew that we could complete the drilling much faster if we drilled holes from the outside, too, so scaffolding was hung from the span. A work platform would be required anyway, when the splice

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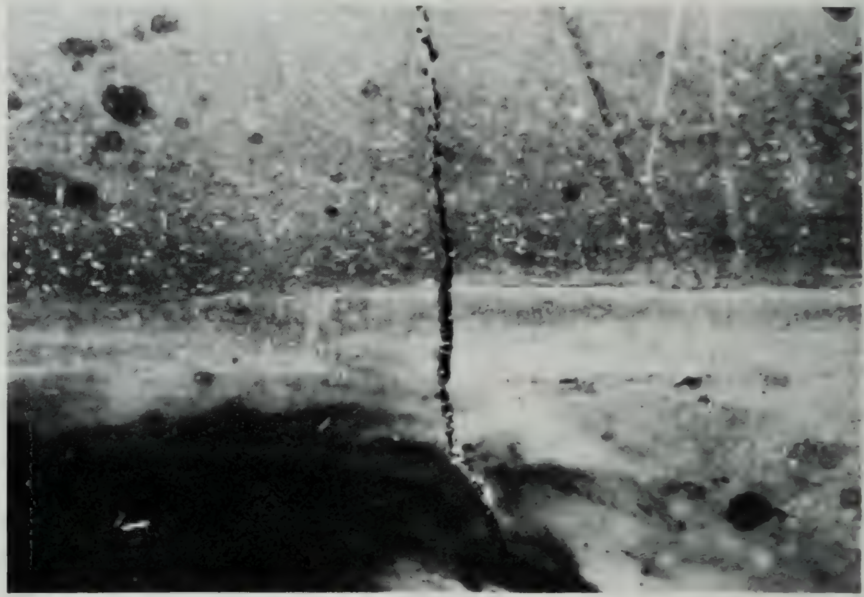

Photo 4. Crack at flange to web weld in span 10.

plates would be applied and the bolts installed. Hole drilling continued through the night and into the next day, with our Bridge and Building forces alternating shifts. In these tight quarters, (Photo 5) it was impossible to employ many drills, but we were able to work as many as three or four drills at one time.

The splice plates began arriving at about sunup. They were set in position and used as templates then, to complete the drilling. The bolting proceeded rapidly behind the drilling operation, and the splice plates were attached in sequence, on the flange, and then on up the web. Most of these bolts had arrived by air freight just that morning. By about 6:30 that evening the splice plates were all secured (Photo 6) with enough high strength bolts to run the first train . . . and so we did.

Our B&B forces had accomplished quite a feat, getting this major structure ready for traffic again in approximately 30 hours. Now, we were finally able to get some much-deserved rest. Our time for R&R would be brief, though. We realized that we still must determine the cause of this crack, and then make sure that it never happens again. The span which had experienced the crack had more redundancy than some of the spans in this bridge, and we might not be so fortunate if this happened again somewhere else in the structure.

We decided to extract a cored sample from the weld at the location where the cracking appeared to have started. The core was then sent off to Lehigh University to be analyzed and a diagnosis rendered. A darker area in the core indicated the cause of the cracking — a weld flaw. A fractographic examination at Lehigh indicated that the crack started from a flaw in a shop weld repair which had experienced cold cracking due to use of improper welding procedures. The actual failure appears to have been a brittle fracture. Since our beautiful bridge has a number of suspect shop weld repairs in the full penetration flange-to-web weld, we have decided to quickly establish a non-destructive testing program which would obviate this type of cracking, and possible failure of the bridge. We are presently engaged in that testing program. The high stress positive moment sections are being ultrasonically tested in priority order. First, those spans in the single track portion of the bridge, where the traffic from both sides of the wye must traverse, are being tested. Then on to the wye portion where each leg carries

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Photo 5. Interior view of crack location.

about one half of the total traffic on the bridge. Strain gages will be employed at strategic locations to aid us in our analysis and decision making. We will be comparing actual live load stresses to calculated ones to determine whether the structure is performing as predicted by theory. If the ultrasonic testing does, in fact, locate flaws in the welds, then we must make a decision. Based on the observed flaw size, can we live with it, or must it be repaired? And if we are to repair the flaw, what type of repair should we use? Welding? Probably not!

Once we have completed this phase of the testing, and made the repairs, if any, we will take another good look at our inspection schedule. Perhaps we will be able to relent from the monthly inspection that is being performed on the entire bridge since the incident occurred. It takes one full week to completely inspect these spans. Regardless of what inspection schedule we choose, you can be assured that it will be *at least* once annually. At Burlington Northern we inspect every one of our bridges at least once annually. The more important ones, and those with a history of repairs are looked at more often. At BN we are proud of our safety record and want to keep it that way.

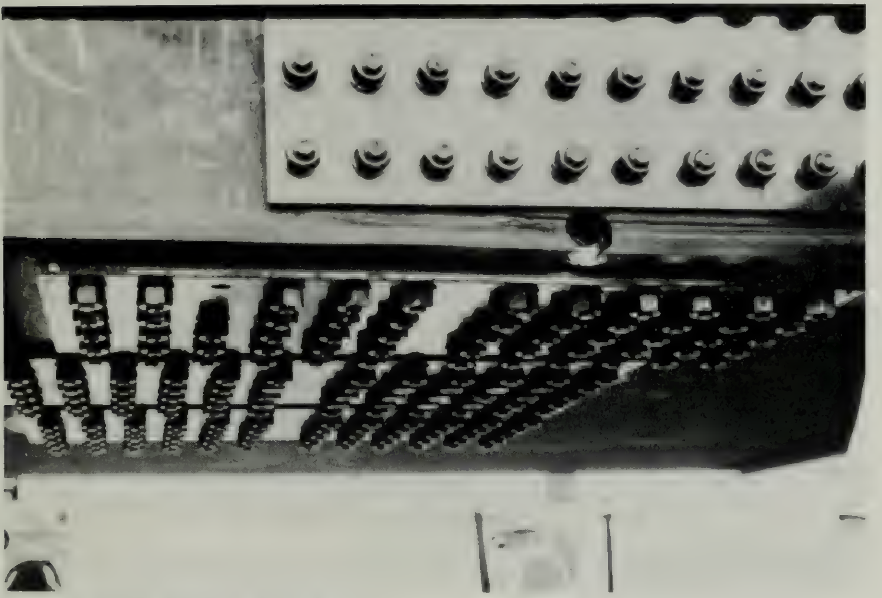

Photo 6. Bolted sandwich splice in place.

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NEW TECHNIQUES FOR RAIL LONGITUDINAL FORCE MEASUREMENT AND RAIL RESTRAINT CAPACITY EVALUATIONS

By: Andrew Kish and Michael Coltman*

1.0 INTRODUCTION

As part of FRA's Track Safety Research program, the Transportation System Center has been conducting studies to improve railroad safety and reduce the number of derailments caused by:

(i) Rail failures due to fracture and fatigue
(ii) Track geometry defects in alignment and crosslevel
(iii) Loss of track lateral stability due to buckling, and
(iv) Gage widening due to inadequate rail restraint

One of the major ongoing research efforts deals with the development of inspection and measurement techniques to detect and control rail and track conditions leading to incipient failures. This paper will focus on two of these techniques, one in the area of track buckling dealing with a measurement technique for rail longitudinal force, and the other in the area of gage widening, which discusses the split axle test vehicle for rail restraint capacity evaluation.

2.0 TRACK BUCKLING AND RAIL FORCE MEASUREMENT

With the increased use of CWR and the recent trends towards heavier axle loads, the prevention of track buckling remains a major concern to the railroad industry. In an effort to improve the safety of CWR tracks, analytical and experimental investigations have been conducted by the Transportation System Center (TSC) supporting the safety mission of the Federal Railroad Administration (FRA). Some of the more recent results of this research include:

- Development of analytical models to predict buckling forces and buckling temperatures
- Conduct full scale buckling tests to study the dynamic buckling behavior and governing parameters critical for buckling prevention
- Development of measurement techniques for track resistance determination and conducting ballast resistivity studies
- Development of safety concepts and criteria for buckling prevention
- Conduct of several rail neutral temperature variation measurements and tests to quantify neutral temperature shift behavior

Some of these research results are currently being used by the railroad industry in improving CWR track stability and preventing buckling included derailments.

The key elements for track buckling prevention include:

1. Ensuring and maintaining good track resistance (both lateral and longitudinal)
2. Controlling line defects and excessive train dynamics
3. Knowing the rail's longitudinal force and its neutral temperature.

Current industry practices and CWR track maintenance specifications adequately handle 1. and 2. above, however, the third still remains a problem in that to date no reliable measurement technique exists for an accurate measurement of longitudinal force in the rail. Hence the rail's neutral temperature (which is an important parameter for buckling prevention) is not known. Therefore the development of a rail force measurement technique/device is of critical importance.

2.1 RAIL LONGITUDINAL FORCE MEASUREMENT CONCEPTS

Several concepts based on various principles of physics and mechanics have been proposed for rail force measurement as summarized in Table 1.

---

*DOT Transportation System Center, Cambridge, MA

Table 1. Summary of Rail Longitudinal Stress Measurement Techniques

TECHNIQUE	COMMENT
Mechanical/optical strain gauge	Adequate for field application, however, needs rail cutting for a reference *"zero"*
Flexural wave propagation	Sensitive to the rail-tie structural damping
X-ray diffraction	Measures surface layer strains only
Acousto-elastic wave	Sensitive to rail microstructure
Magnetic coercion	Sensitive to rail microstructure and residual stress
Barkhausen noise	Very difficult under field conditions
Electromagnetic-acoustic transducer (EMAT)	Sensitive to rail microstructure and rail surface condition
Laser *"spackle"*	More useful in lab application due to accuracy required for mapping laser interference pattern

Although these techniques are based on valid theoretical concepts, they have the following limitations and shortcomings for practical application:

- An inability to measure absolute rail force *(i.e. these techniques require an initial reference level which is usually obtained by rail cutting and destressing)*
- Very sensitive to rail microstructure, rail residual stresses, and rail surface conditions *(to a point where measurement accuracy may be seriously compromised)*

To address these problems, a new technique has been recently developed and a prototype car mounted device has been used to validate the technique through field tests. The technique is based on the well-known beam bending principle of mechanics and it measures the absolute rail force. It is not *"destructive,"* but requires removal of spikes and anchors from the rail to be tested.

2.2 RAIL UPLIFT METHOD

If the rail is freed from ties over some length, restrained vertically at the ends of the freed portion and subjected to a concentrated uplift load at the center, the resulting deflection depends on the magnitude of the rail longitudinal force. Clearly, longitudinal compressive load will increase the deflection of the *"beam-column,"* and tensile force will reduce it. For a given length of rail, the vertical force required to produce a specified deflection is a measure of these rail forces. The concept implementation is based on the fact that the rail can be conveniently held at the two points by the wheels of a rail car. This automatically fixes the length of the rail and provides the proper restraint conditions at the ends of the rail beam. The spikes and anchors between the inner wheels of the two trucks of the car must be removed. Fig. 1 shows schematically the rail uplift method; Fig. 2 shows the rail car mounted hydraulic fixture lifting the test rail.

An analytical model has also been developed to calculate the vertical deflection produced by different levels of rail force. This model showed that this vertical deflection is measurably sensitive within the range of longitudinal forces of interest in rail neutral temperature determinations. For example, Fig. 3 shows the influence of rail size on the uplift force required for different levels of longitudinal force. As can be seen, the uplift force vs longitudinal force is linear with a well defined slope which is useful for test technique calibration.

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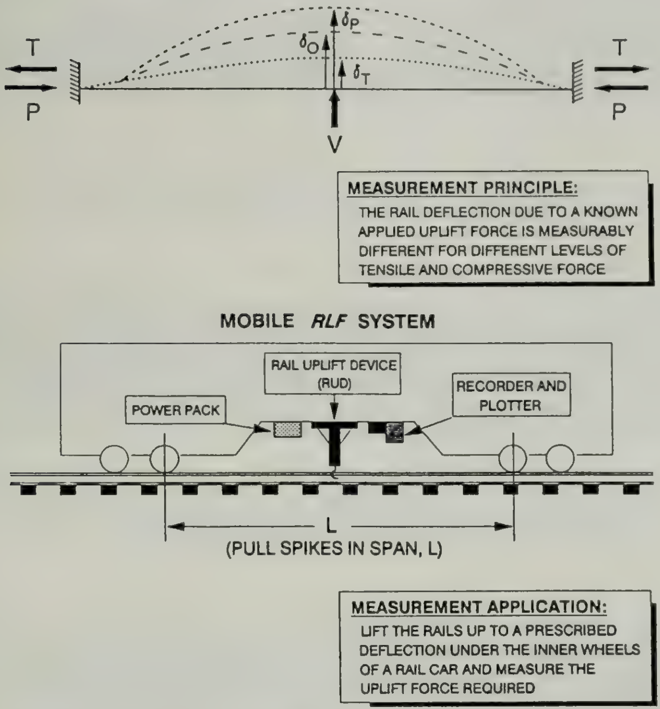
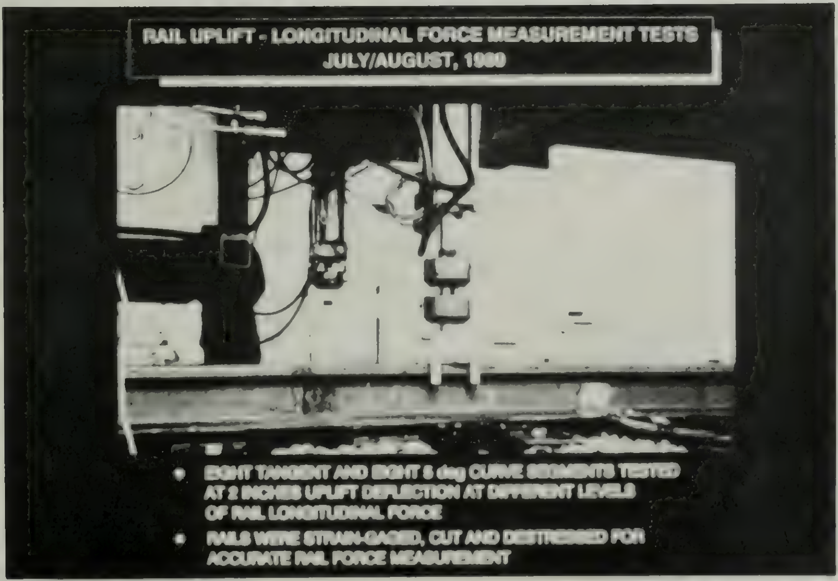


MEASUREMENT PRINCIPLE:
THE RAIL DEFLECTION DUE TO A KNOWN APPLIED UPLIFT FORCE IS MEASURABLY DIFFERENT FOR DIFFERENT LEVELS OF TENSILE AND COMPRESSIVE FORCE

MEASUREMENT APPLICATION:
LIFT THE RAILS UP TO A PRESCRIBED DEFLECTION UNDER THE INNER WHEELS OF A RAIL CAR AND MEASURE THE UPLIFT FORCE REQUIRED

Figure 1. Rail uplift concept and measurement principle.


RAIL UPLIFT - LONGITUDINAL FORCE MEASUREMENT TESTS
JULY/AUGUST, 1989

- EIGHT TANGENT AND EIGHT 5 deg CURVE SEGMENTS TESTED AT 2 INCHES UPLIFT DEFLECTION AT DIFFERENT LEVELS OF RAIL LONGITUDINAL FORCE
- RAILS WERE STRAIN-GAGED, CUT AND DESTRESSED FOR ACCURATE RAIL FORCE MEASUREMENT

Figure 2. Rail uplift device.

2.3 TEST RESULTS

To verify this rail uplift measurement technique, tests were conducted at the Transportation Test Center in Pueblo, CO on a tangent and 5-deg curved track segment. An existing track loading vehicle with truck center spacing of 37.5 ft. was redesigned to provide the rail uplift capability by mounting a hydraulically actuated uplift device containing a load cell to measure the uplift load. The test section rails were instrumented with force measuring strain gauges and the rails were cut and destressed for an accurate rail force determination. Different levels of longitudinal force was achieved by rail heating. Eight tangent and eight five degree curve segments were tested, and the rail force was correlated with the vertical load required for a 2-in. rail uplift deflection.

Figure 4 provides an example from the tangent test results, showing the well defined linearity, and slope. In general the tests showed:

- Good results on both tangent and curved rail segments, with an accuracy range of ± 10-20 kips longitudinal force *(which is equivalent to a ±4-8 deg-F neutral temperature change)*
- For curved rail segments, measurement concept/car design needs to be adjusted for different vertical wheel loads on the high and low rails due to superelevation.
- Excellent agreement with theoretical predictions

In conclusion, in view of these results and analytical studies, the proposed technique based on the rail uplift-bending response appears to provide a viable method for rail longitudinal force measurement and neutral temperature determination. Additional validation tests are planned at TTC this summer to finalize the technique accuracy and in-service applicability.

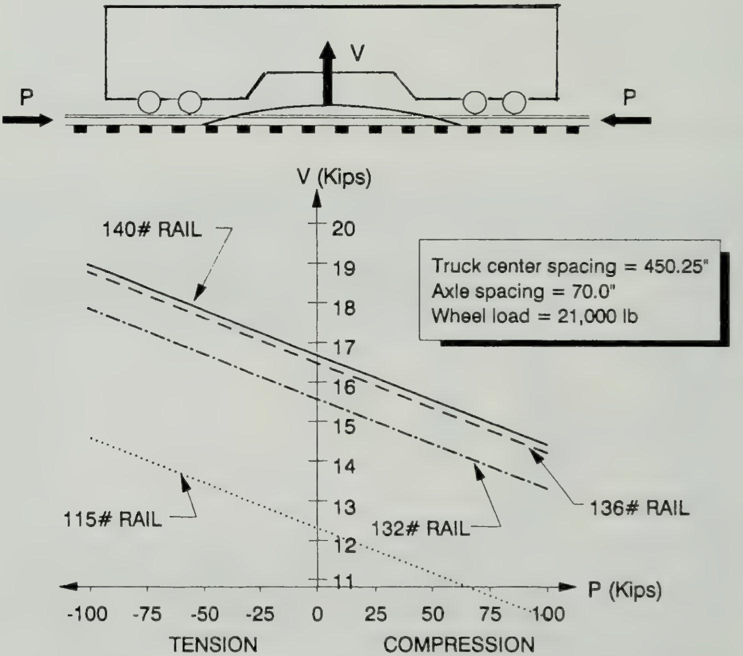


Figure 3. Rail size influence on uplift force vs longitudinal force for a 2.0 inch uplift deflection

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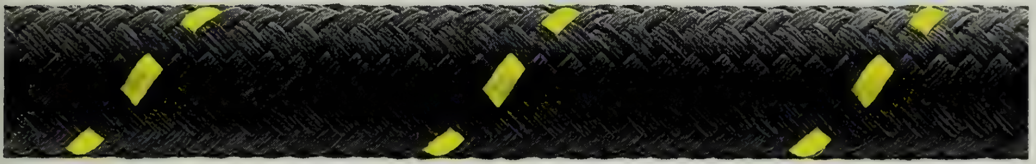

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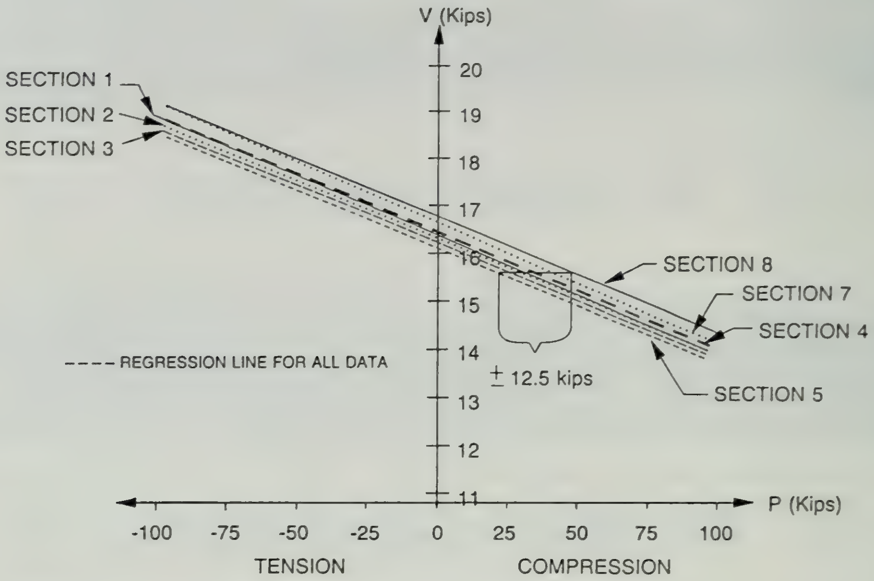

Figure 4. Tangent uphill test results on eight test segments at TTC

3.0 RAIL RESTRAINT CAPACITY EVALUATIONS: SPLIT-AXLE TEST VEHICLE

Another important mode of track failure is gage widening resulting from high lateral loads occurring at locations with weak tie and fastener conditions.

While the absolute number of derailments has decreased, sudden gage widening continues to be a major cause of derailments accounting for over 20% of the track caused accidents. With the trend toward higher density mainlines and heavier axle loads the tie and fastener structure can be expected to degrade at an increased rate.

Ties are also a major investment requiring the industry to install around 20 million cross ties a year. So the FRA's research goal has been to develop an inspection technique which can identify critically weak tie conditions and provide key performance data to improve the efficiency and cost effectiveness of tie renewal programs.

The key elements to eliminating gage widening are:

1. Control force generation mechanisms such as track geometry irregularities or train action.
2. Control Track Gage
3. Identify weak tie and fastener conditions.

The TSC approach has been to investigate both the load generation mechanisms and actual tie strength to develop rail restraint requirements and inspection techniques.

In the area of tie strength evaluation, a number of tests were conducted where the track was subjected to increasing lateral loads while the rail head and base deflections were monitored. The tests included variations in vertical loads and rail weight. The results provided a basis for describing generic strong and weak tie characteristics. Shown in figure 5 are composite curves of the rail head displacements measured under 15 and 30 kip vertical loads during some of these tests.

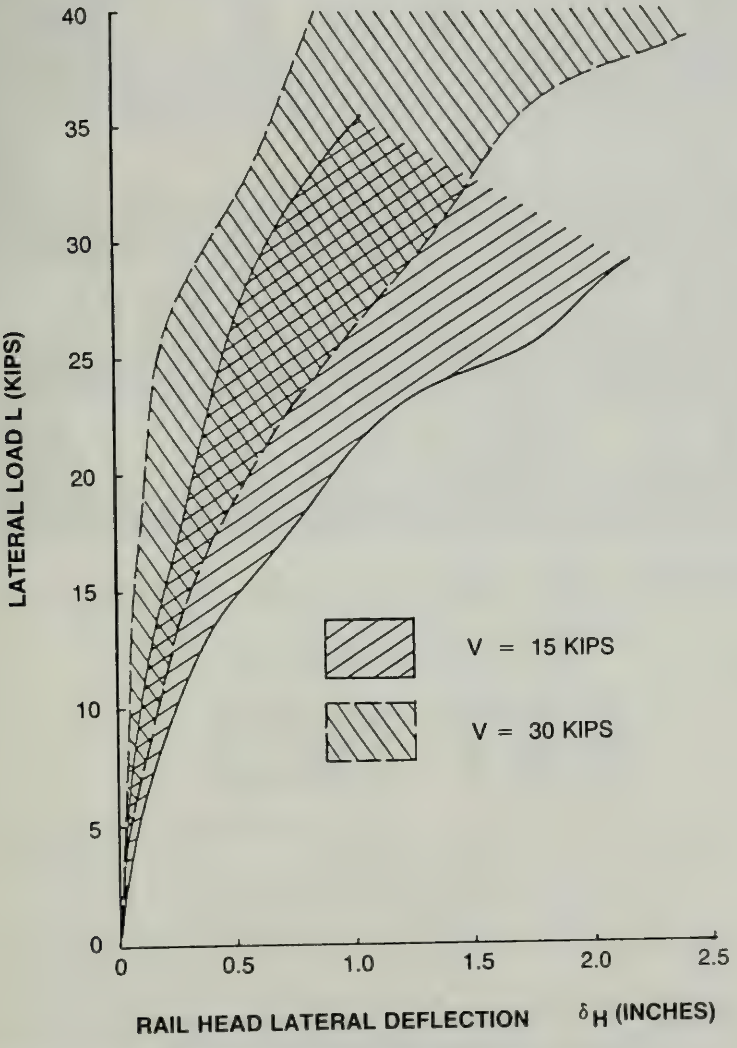

Figure 5. Lateral Load Versus Railhead Lateral Deflection Summary

The other part of the problem was to determine the vehicle induced loads the ties should be capable of supporting. Through a series of vehicle/track interaction tests various track geometry irregularities were constructed to determine the range of lateral wheel loads possible. Once unacceptable track geometry deviations were eliminated, the peak lateral wheel forces were seen to be in the range of 25 kips for 33 kip wheel loads, and relatively insensitive to speed.

With the limits on the vehicle induced loads defined, and an understanding of tie strength characteristics, a performance based inspection technique could be developed for defective tie conditions.

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Before discussing this performance based technique, a review of current inspection procedures is useful. Gage restraint is evaluated by a combination of several procedures. First, many railroads use Track Geometry cars to identify critical defects. By eliminating these, the railroads eliminate the most critical force generating mechanisms. In addition, segments with certain patterns of irregularities are assumed to have weak tie conditions. A geometry car, however, can't find these weak spots until they manifest themselves in some geometry deviation.

Next, track is surveyed by high rail vehicle which provides an overall subject assessment of track condition. However, significant weak tie conditions can be missed.

Finally there is the track walker's inspection, which is quite slow and provides only a subjective assessment. The problem here is that numerous ties that look strong are weak and vice versa. The Gage Restraint Measurement System, shown in figure 6, provides a performance based evaluation of rail restraint capacity by directly loading the track and measuring the amount of gage widening. This provides a continuous measure of tie strength which is directly correlated to gage. Further, the split axle design can provide the information at 25 mph.

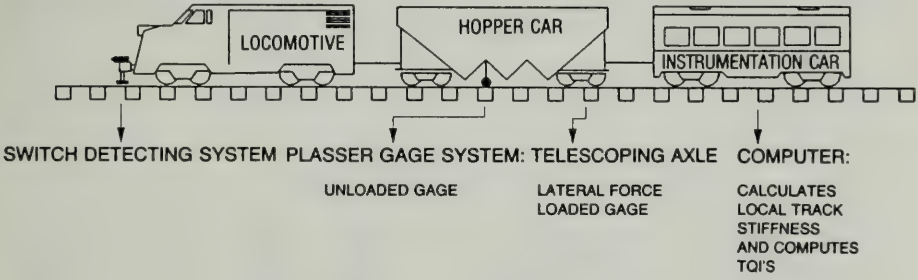

Figure 6. Gage Restraint Measurement System

The split axle shown schematically in figure 7, is a normal freight axle, cut in half and held together with a large steel sleeve. The axle is equipped with hydraulic actuators to provide gage widening forces on the rail. The heel plates are strain gaged to provide a measure of the applied lateral loads. The split axle is equipped with a displacement transducer to measure gage directly at the axle. This is compared with the nominal gage providing a measure of gage widening.

With the nominal gage, and the amount of gage widening caused by a lateral load, three indices have been developed as descriptors of the ties and fastener condition. Two are based on predictions of maximum dynamic gage under different severe loading conditions, and the other is based on a direct stiffness measurement. These indices have been developed and evaluated in over 2000 miles of testing on seven railroads. Current test programs are underway on CSX and the UP evaluating specific thresholds for the indices and degradation rates.

Shown in figure 8 is an example of the data obtained during normal survey. The top two traces are unloaded and loaded gage, the next three are the indices used to evaluate tie condition. This was an interesting stretch of track in that there is a bridge here and we see two weak track locations, one at each end. Most importantly, the location on the east end "did not look" especially weak.A detailed visual inspection of the location was difficult because the tie and fastener are obscured by the guard rails. The spot, however, was one of the weakest spots in the track segment.

To summarize, the split axle or gage restraint measurement system's development is complete. Research continues only in the areas of track degradation and the definition of absolute safety limits.

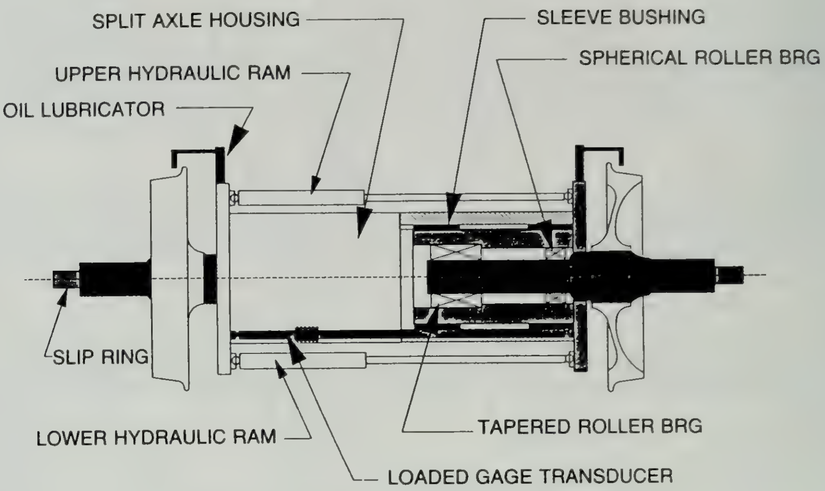

Figure 7. Cut Away Schematic of Loading Axle.

The equipment consistently finds the weakest tie conditions in any track segment and it discriminates between ties that just look weak and those that are weak even though they look fine. The equipment can survey between 100 and 200 miles per day at 25 mph, and most importantly the data can be used to:

1. Prioritize major tie programs by providing objective data to compare the overall strength of widely different line segments.
2. Optimize spot maintenance forces — i.e. when a major program has to be postponed, the split axle can identify critical locations in need of short term attention.
3. And finally, the data can be used to develop long term maintenance requirements by providing estimates of the numbers of ties needing replacement and the approximate period before a major tie renewal program should be scheduled.

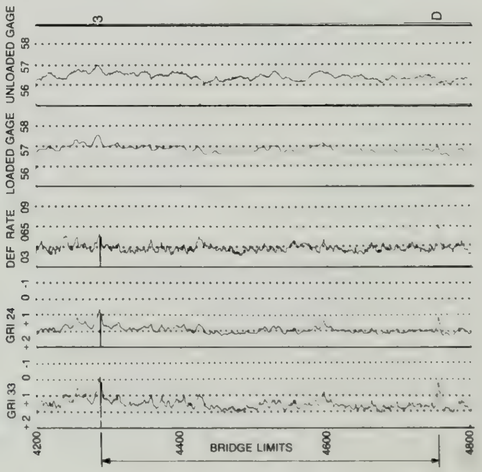

Figure 8. Example System Survey Output.

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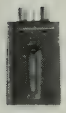
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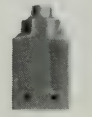
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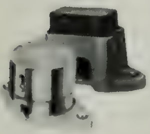

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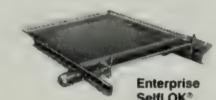
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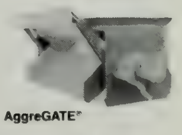
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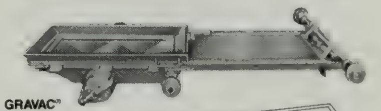
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USE OF PREFABRICATED COMPONENTS FOR THE RECONSTRUCTION OF BRIDGES UNDER TRAFFIC ON THE MEXICAN RAILWAYS

By: Ing. Eduardo Ramirez Cato*

Introduction

The use of prefabrication in construction has been known since the early days of mankind, when in order to build houses man substituted rocks for dried mud bricks. This element was the first one to be prefabricated with constructive purposes.

Since that time until the past century, prefabrication did not progress significantly until Joseph Aspadin discovered Portland Cement in 1824. Since then, brick production became sophisticated and concrete started to be used either alone or combined with steel. It was in this century when big steps on prefabrication were made, first with precast concrete slabs and later with almost all kinds of prestressed concrete elements. Nowadays practically no construction can be understood without the use of prefabricated elements.

Completed prefabricated component bridge used in replacement of a timber trestle.

In the same way, steel prefabrication developed at a fast pace during the nineteenth century, when Bessemer improved steel smelting and as a result there was no limit to the size of constructions that could be made. Due to the advantages it has, all kinds of projects were carried out with steel: railroads, buildings, factories, bridges and even very impressive monuments. Construction of railroad bridges has also benefitted from this technological progress. Prefabricated elements are being used advantageously, especially when reconstructing old structures under traffic.

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*Assistant Commissioner Bridge Section, National Railways of Mexico

Main Advantages of Using Prefabricated Components in Bridges on the National Railways of Mexico

The main advantages of using prefabricated components for railroad bridge reconstruction are the following:

Operating Conditions

It's highly desirable to reduce as much as possible the time a bridge is taken out of service for repairs or reconstruction, so that train traffic is disturbed as little as possible. By applying prefabricated components, certain preliminary building processes can be made outside the track, and only when it is necessary, the track is occupied for a short time.

Gang Safety

Prefabricated elements are designed so that they are easily handled, according to the overall construction process. This reduces hazards during the work.

Best Quality

By prefabricating elements in a shop, the risks of utilizing defective materials or wrong construction procedures are decreased. With prefabricated elements, optimum quality is assured for each part of the bridge.

Handling

Within the logical restrictions that a bridge design imposes, each of the elements is constructed according to the equipment that the railroad or its contractors have available, so that, they can be handled in the easiest, fastest and safest way. In this way it is possible to obtain the best results during a bridge construction.

Savings

Even though there is no material cost difference between purchasing prefabricated structures and the cost of making them on site, savings are obtained from the increase in productivity during construction, better quality and low maintenance costs.

Construction Experiences in Prefabricated Bridges on the National Railways of Mexico

Like most railroads since the beginning of the concrete age, the National Railways of Mexico has used a wide variety of prefabricated elements as a part of bridges. The first prefabricated elements used by the National Railways of Mexico were the precast concrete slabs, which are still being used nowadays on sizes varying from 6′6″ to 21′6″. These slabs are cast as near as possible from the bridge location in order to make distribution and mounting easier. On the National Railways of Mexico there are 3,500 bridge superstructures with this type of slab.

Precast concrete elements have also been used for culvert construction with excellent results. In these cases square section pipes made of reinforced concrete are utilized. They are prefabricated near the working place and installed with a crane. In Mexico there are more than 200 culverts built in this manner. Due to the new technologies and in order to lighten the parts of this type of structure, the same pipe can be made out of prestressed concrete.

During the fifties, when the use of prestressed concrete was introduced in railroad bridge construction a whole new field was opened for these elements employing the postensioned technique. In order to rebuild bridges with a superstructure in prestressed concrete, we apply almost exclusively prestressed components made in a plant. The beams used in this type of bridge construction vary from 23′ to 51′ in length. They are box type beams with similar design to the ones recommended in previous editions of the A.R.E.A. Manual. Each span is made up of 4 or 5 box beams, which once mounted are postensioned transversally to integrate the span. There are 1,500 bridges in the National Railways of Mexico built in this manner.

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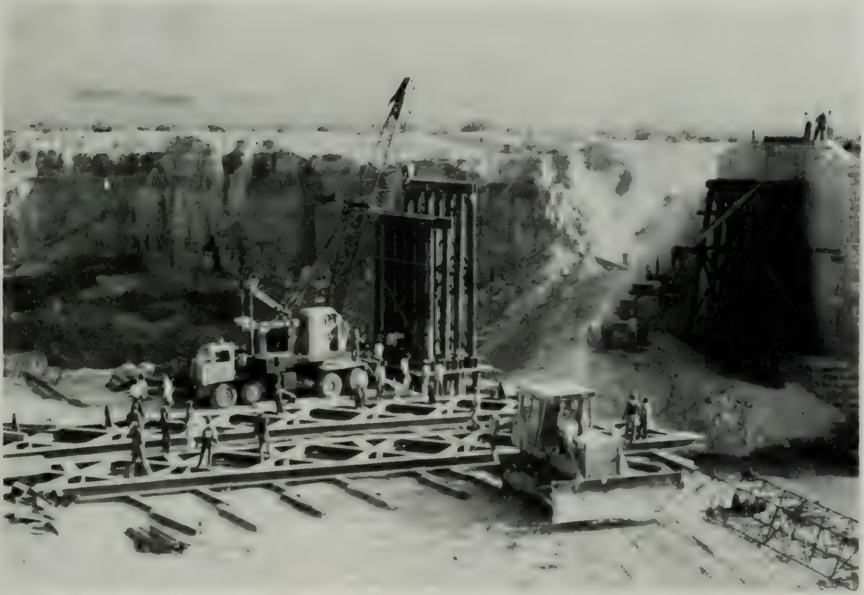
Referring to substructure, in order to preserve train traffic with the least possible disturbance, prestressed concrete piles have been driven, each time more frequently, The standard piles are a square section 24″ x 24″, and for an octagonal section we have established 18″ diameter piles. At present, we have 2,000 bridges with prestressed concrete pile trestles. The Mexican experience shows the advantage of driving prestressed concrete piles instead of reinforced concrete ones.

Our experience and the good performance shown by prestressed concrete elements, led us to try and find the best construction process so as to reduce time, equipment and manpower required to rebuild a complete bridge. Later on, I will explain more in detail the process and elements used for this purpose.

Regarding metal structures, steel bridges appeared during the past century, before concrete structures were born. The smallest beam or the most complicated truss may be considered as a prefabricated structure. Once it is brought to a bridge location it is put in place by means of a crane if a beam, or assembled near the bridge when we are dealing with a truss, and then installed using any of the different launching procedures. This method has been applied continuously for replacement of trusses, and with the equipment available on the National Railways of Mexico, it has given excellent results. At this point it is important to emphasize that the metal elements were born to be always prefabricated.

Another significant experience in solving emergencies derived from bridge washouts is the use of modular structures in order to pass over spans as long as 150′. The modular trusses are assembled on site and may be reused once they are replaced by a permanent structure. This type of truss (see photos on pages 251 and 252) was adapted from the Bailey type and we are looking forward to optimizing it as a deck bridge or as a through-pass bridge, whichever is required.



Construction of a modular truss on site.

In false work construction, a commonly used superstructure has been the steel duplex beam, with integrated track between each pair of beams. This type of structure allows important time savings.

Installed modular truss for bridge replacement.

As a complement to the latter superstructures used in bridge emergencies, it is convenient to use modular substructures that do not require sophisticated mounting equipment and allow the use of different-size spans, bearing the structure directly on the ground.

In the Mexican National Railways we have been using metallic structures since the time the first lines were built in the last century and are still being widely used during our time. Metallic structures are more suitable for bridge reconstruction when work has to be performed under traffic, than postensioned concrete structures. The usual lengths for steel structures, vary from the smallest 13′ beams laid over timber bents or masonry substructure, to the 500′ trusses utilized in some bridges over important rivers and viaducts. The National Railways of Mexico has 5,500 bridges with either metallic trusses, through-pass beams or deck girders. A large number of the metallic structures in our railroad were built with the original railroad lines during the past century, and they are still heroic witnesses of train traffic, even though the weight of the trains has considerably increased compared to the loads for which they were designed.

The combination of steel and concrete has also given very positive results, principally in metallic structures on long, high spans, basically because of the elimination of fire risk that in many cases causes irreversible damage to the structures. For this reason we in N. de M. are increasingly using prefabricated prestressed concrete slabs in composite construction with metallic beams and trusses. When the steel structures are already working on site the slabs are cast at a different location and then mounted on the floor system. The ballast section and concrete ties are then laid, thus providng a uniform finish to the rehabilitated line.

A New Whole Prefabricated Design

The Mexican National Railways are using a new system to construct prestressed concrete bridges, where all the elements that constitute them are prefabricated in a similar way as recommended in past editions of A.R.E.A. Manual. We manufacture each element in a shop: piles, caps, beams, and wing walls. The elements are then sent to the working site and assembled together until the bridge is finished.

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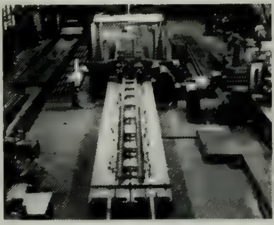

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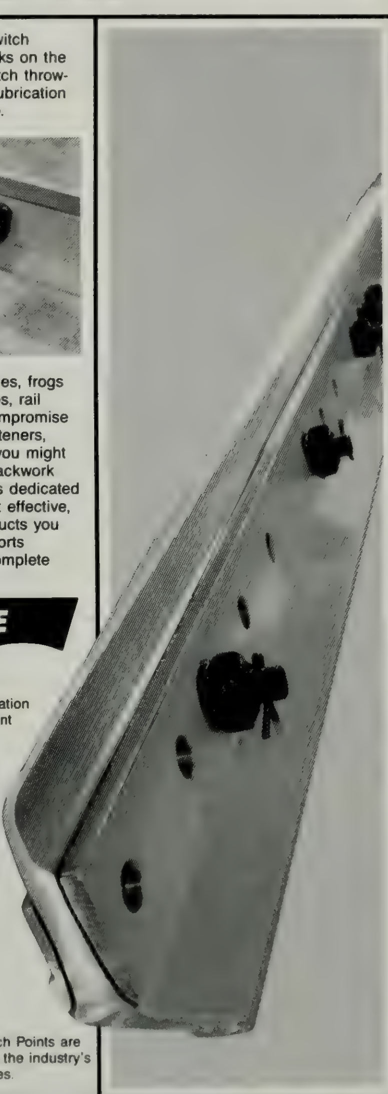

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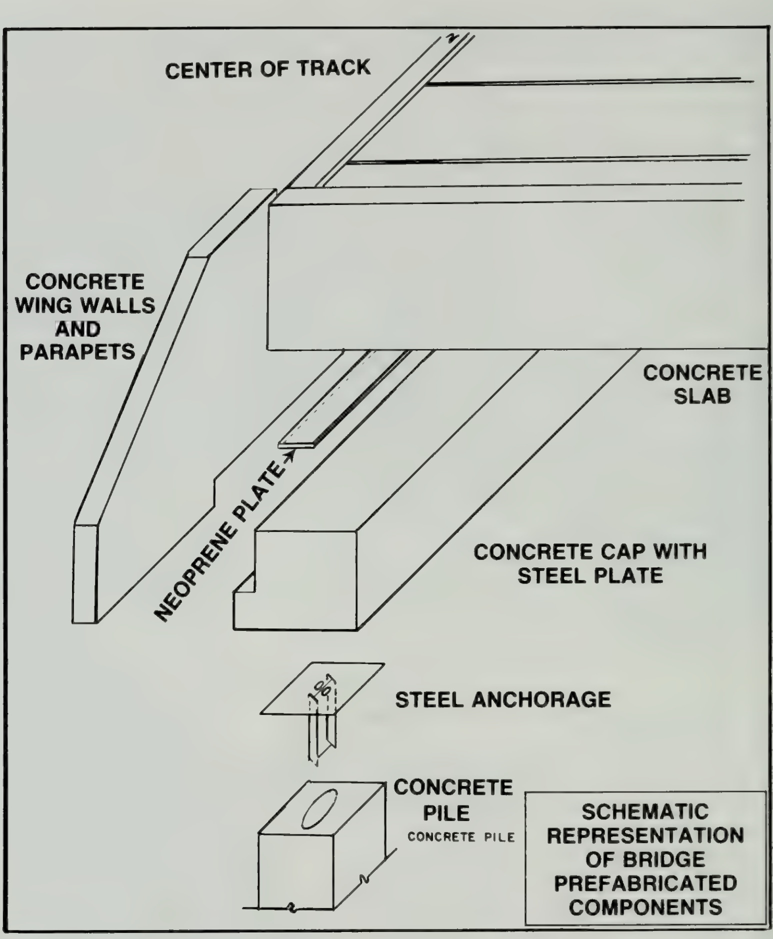
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CONCRETE SLAB
NEOPRENE PLATE
CONCRETE CAP WITH STEEL PLATE
STEEL ANCHORAGE
CONCRETE PILE
CONCRETE PILE
SCHEMATIC REPRESENTATION OF BRIDGE PREFABRICATED COMPONENTS

Schematic of components in concrete prefabricated bridge.

Either octagonal or square section piles are used by the Mexican Railways. Both types of piles are lightened by a cardboard or polystyrene core especially designed for this purpose. Once the piles are driven, it is necessary to cut them. The cutting level must be as required for mounting the caps. This cutting edge must reach the hollow part of pile where anchors will be cast. Another possibility we are trying to develop for this type of project is the use of steel pipe piles or "H" beams as substitutes for the prestressed concrete piles. In this way the welding of the caps to the piles would be much easier.

Once the piles are cut, a small pouring is required to fix the metallic anchors that will support the cap. The cap is a light prestressed concrete element, having also a 1/2" thickness plate on its lower part so that it can be welded to the previously mounted anchors. The extreme caps were those properly designed for mounting the wing walls and the intermediate caps to support two adjacent spans.

It is important to mention that all these operations may be effected at times when trains do not run. It is not necessary to disrupt traffic or cut track for any of the previous steps, thus allowing the work to be made in ordinary journeys, achieving in this way important savings.

The next step is to set the 3/4″ neoprene bearing pads to reduce impact. It will be necessary to have track time without any circulation of trains, so we can take away all the elements of the false work or old bridge and proceed to install the concrete beams. To mount the beams and also to handle the other parts of this bridge we work with the same 77 kip crane used to drive the piles. This type of crane can easily handle jobs for spans up to 51′.

To finish the job it is necessary to place wing walls, also made of prestressed concrete, in each of the end caps of the bridge to retain the material of the embankment. So that all the elements work properly, it is necessary to apply postension between the beams as well as the wing walls against the caps. For this purpose holes are provided to accommodate the tensioning cables. Track laying and ballasting is done like in any other standard stretch of track.

This method has been applied successfully for replacing timber trestles and it represents a very good tool for future programs. Traditionally, the construction of bridges in the case of trestles of concrete piles and slabs is done by casting the caps and wing walls on site. The reinforced concrete slabs are cast at a nearby location and are mounted afterwards.

Installation of prestressed 24 ″ square concrete pile in preparation of bridge renewal.

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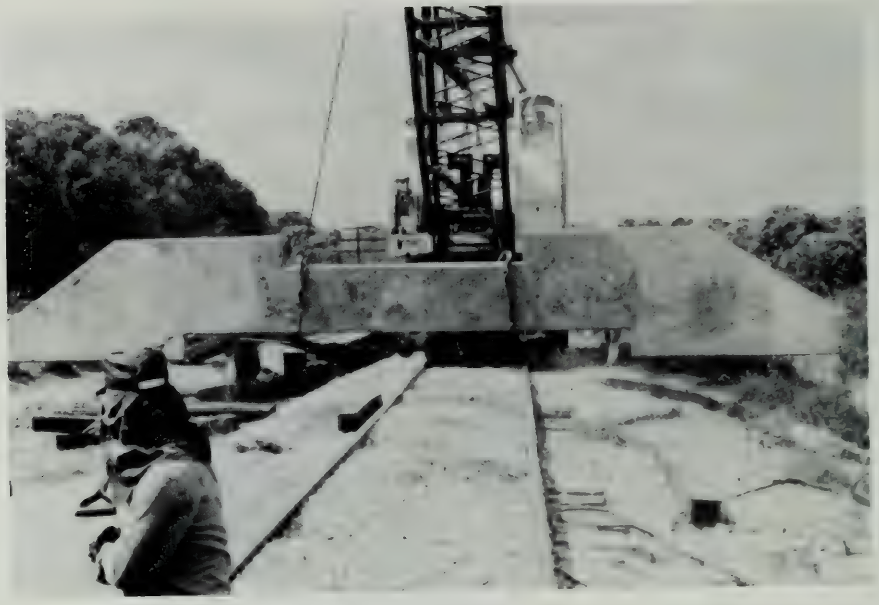
Placement of prefabricated components.

Comparing the traditional procedure with the new one, we can see that the construction time for a bridge in the field is drastically reduced, especially with the use of prefabricated caps and wing walls. The following table shows the main construction items of a bridge and the time required in relation with the whole project. In this case we are talking about a 30′ span with two 3 pile bents. The table shows the comparison between one method and the other.

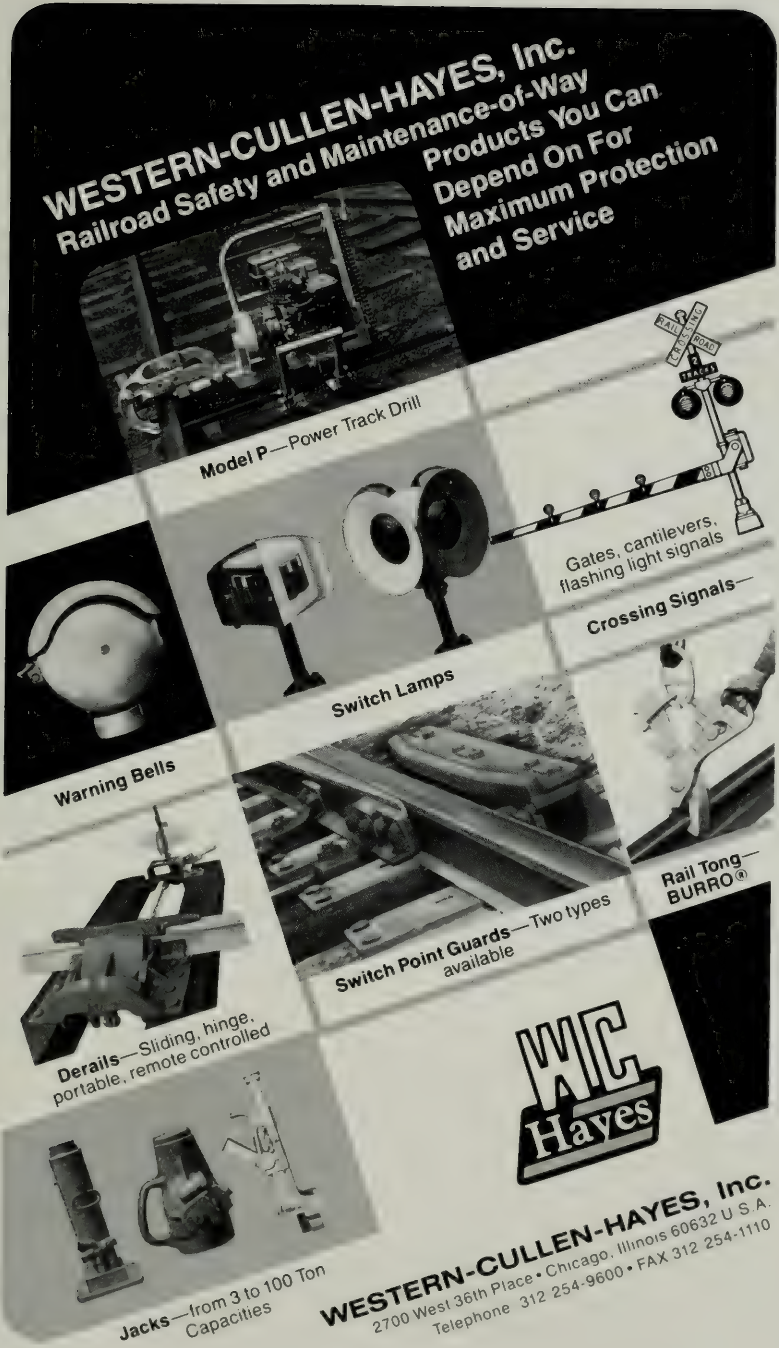
Time Required for a 30′ Span
Bridge Construction with 3 Piles Each Trestle

	Item	Traditional Way	New Design	Difference
Sub-	Piles Driving	3	3	0
Structr.	Caps	39	3	36
	Wing Walls	11	1	10
Super-	Setting-up	5	5	0
Structr.	Track Laying	2	2	0
		60 Hrs.	**14 Hrs.**	**46 Hrs.**



Prefabricated wingwall being moved for placement.

Track installation on prefabricated spans.



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With this method we can build a bridge of three 30′ spans in only 45 hours. Following are the principal items, installed with a 15 people gang:

Main Items for a 3 x 30′ Spans
Bridge Construction

Item	Quantity	Time (Hours)
Pile Driving	12 SQR 24″ x 24″	12
Anchors Bucking	12 (One for each pile)	9
Caps Setting-Up	4 Pcs. 16′ x 3′3″ x 2′7″	2
Caps Welding	4 to the anchors	6
Slabs Setting-Up	3 Slabs of 4 elements each	6
Wing Walls Setting-Up	2 (One for each approach)	1
Posttensioning	9 on slabs, 6 on wing walls	3
Track Laying	93′ over the bridge	6
		45 Hrs.

The figures permit having a very clear idea about the advantages that this system provides in the case of a bridge destruction caused by washouts.

Outlook for the Use of Prefabricated Elements in the Construction of Railroad Bridges

In my opinion the use of prefabricated elements is necessary today as will be in the future to solve construction problems in railroad bridges, because it is evident that we obtain a much higher performance with their application. Because the Mexican National Railways has many bridges that require substitution due to low capacity or deterioration, the use of prefabricated elements looks very promising.

If we continue at the pace of 200 new bridges built each year, we shall build 2,000 new ones within the next 10 years. Out of these 90% will have prefabricated elements of kinds already described in this paper, or possibly utilizing new better materials which may in the future be developed.

PUBLISHED AS INFORMATION BY COMMITTEES

COMMITTEE 4—RAIL

Report of Subcommittee 3 Rail Statistics

Subcommittee Chairman: A. E. Shaw, Jr.

Consolidated Report of Rail Shipped To North American Railroads from North American and Non-North American Producing Mills in 1988 By Weight and Section

Weight	Section	N. American Tons Shipped	Non-N. American Tons Shipped	Total	% Total
100	RA	4,247	0	4,247	0.6
100	RE	0	0	0	0
115	RE	54,291	32,461	86,752	12.9
119	RE	12,088	0	12,088	1.8
122	CB	2,158	1,046	3,204	0.4
132	RE	154,677	64,878	219,555	32.7
133	RE	83,707	14,034	97,741	14.6
136*	RE	213,697	32,395	246,092	36.8
140	RE	243	0	243	—
OTHER			1,289	1,289	0.2
TOTAL		525,108	146,103	671,211	100

*Includes 136# with modified head profile

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It's as true today as it was over one hundred years ago: there's nothing like wood for crossties.

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COMMITTEE 8—CONCRETE STRUCTURES AND FOUNDATIONS

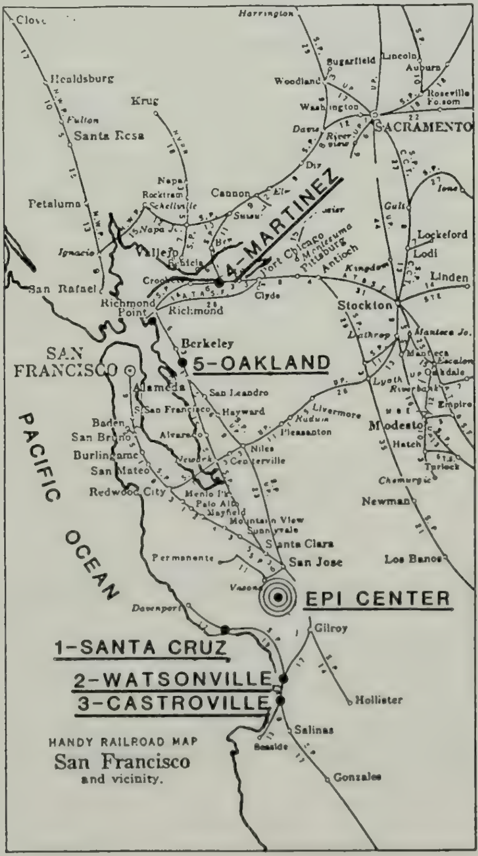
Chairman: D. J. Lewis

RAILROAD BRIDGES IN THE LOMA PRIETA EARTHQUAKE

Subcommitte Chairman: T. R. Kealey

On October 17, 1989, the Loma Prieta Earthquake struck San Francisco, California, and the surrounding area. This quake has been reported to be of magnitude 7.1 on the Richter Scale, with an epicenter near the Town of Santa Cruz some 65 miles southeast of downtown San Francisco. The damage to highway bridges, buildings and utilities has been well documented and reported. Because there was no major damage to railways and railway structures, many of which are in relatively close proximity to some severely-damaged highway structures, they received little publicity, but what effects there were are useful to document.

Committee 8, AREA, has been concerned about the seismic design and retrofit of railway structures since a seismic design guide for highway bridges was prepared for the Federal Highway Administration in 1981. Contact has been maintained with railroads in the seismic areas in California, specifically with the Southern Pacific Transportation Company and the Atchison, Topeka and Santa Fe Railway Company. Effects of the earthquake have been observed on four bridges of the former, and one bridge of the latter, operated by the Oakland Terminal Railway. The locations of these bridges and their relation to the epicenter are shown below. There follows a detailed description of the effects of the earthquake on each.



Southern Pacific Transportation Company's Bridges

1—Bridge 119.67 - Santa Cruz (13 ± miles to epicenter)
2—Bridge 101.26 - Watsonville (12 ± miles to epicenter)
3—Bridge 113.46 - Castroville (25 ± miles to epicenter)
4—Bridge 33.31 - Martinez (65 ± miles to epicenter)

Atchison, Topeka and Santa Fe Railway Company's Bridge

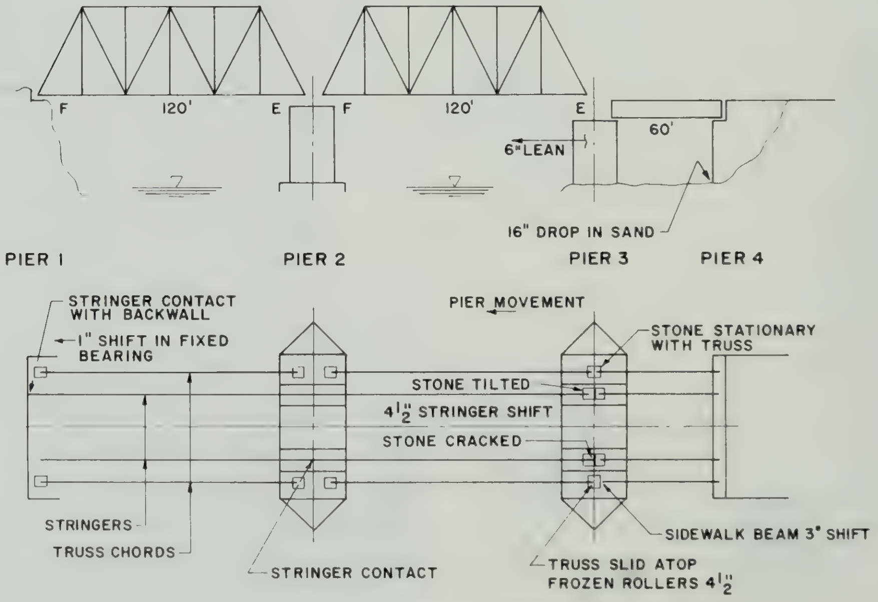
5—Bridge 11.65 - Oakland (50 ± miles to epicenter)

1—Bridge No. 119.67 - Santa Cruz

This bridge, constructed in 1903, consists of two 120-foot long through riveted truss spans and a 62-foot long deck plate girder span. The west end abutment (Pier 1) of the bridge is in a rock hillside, the main piers are in the San Lorenzo River and the east abutment (Pier 4) is on a soil embankment. The bridge is located about 200 yards from the ocean and about 13 miles from the epicenter. Pier construction consisted of pile-supported concrete stems with cap stones. The truss stringers are independently supported on the piers on granite bolsters.

Earthquake effects were generally confined to Pier No. 3, which apparently tipped about 6 inches toward the river channel. On the ocean side of this pier, there were several prominent depressions in the sand in line with the pier, which, after several days, had partially filled in from wave action at high tide. Atop this pier on the north side, the cap stone shifted eastward 5 inches relative to the pier concrete, and the adjacent stringer bolster was tilted as a result of the bottom lateral hanging up on the cap stone. The cap stone which was well anchored to the truss stayed with the truss, while the pier rotated to the west. The cap stone and frozen roller nest on the south side rotated with the pier. This exposed 4-1/2 inches of the end rollers. The adjacent stone bolster for the stringer cracked at the anchor bolt. To the outside, a single independent girder supporting a sidewalk had a 3-inch shift at its bearing. There was no damage at the east abutment, but the beach sand adjacent to the concrete pier dropped 16 inches.

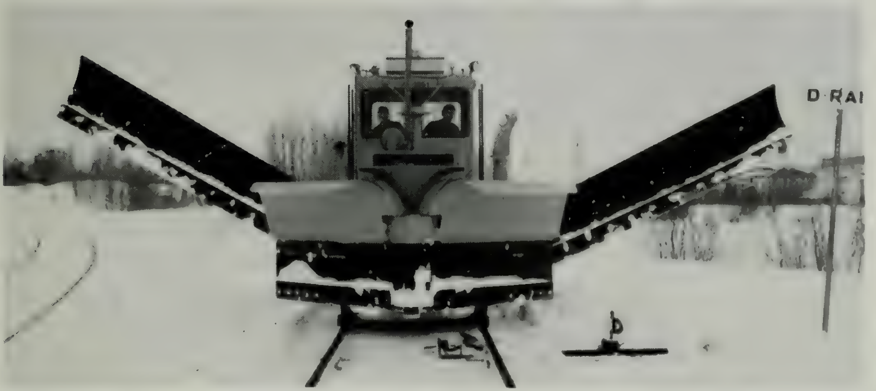
The shifted stones will be secured and grouted. Monitoring of the tilted pier over a period of time will determine if the pier needs to be further stabilized; however, no other remedial action is proposed at this time.



Bridge No. 119.67

PYKE

BALLAST REGULATOR
&
SNOW CLEARING MACHINE
MODEL M



Model M Snow Clearing Machine shown with 16 foot reach hydraulically-operated snow wings, multi-position front snow plow, and snow blower, all cab-controlled.

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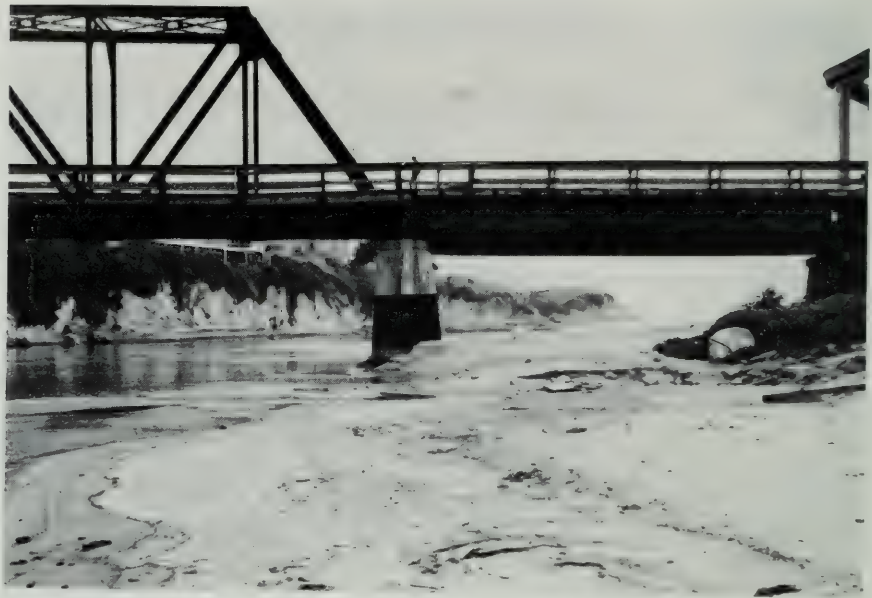
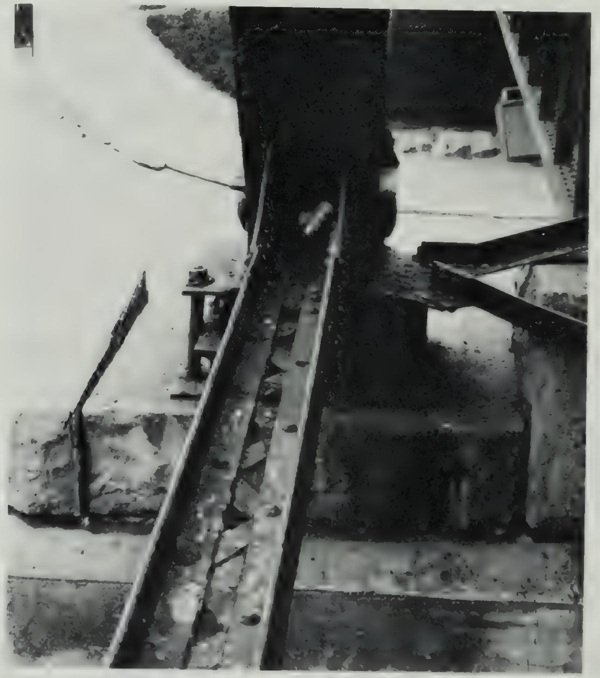


View of Pier 3, Bridge 119.67, tipped six inches to the west.



View of the left side of Pier 3, Bridge 119.67, noting the shift between cap stones and concrete stem.

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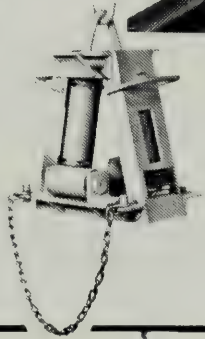

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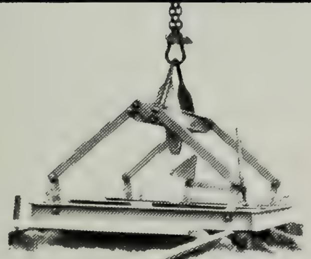

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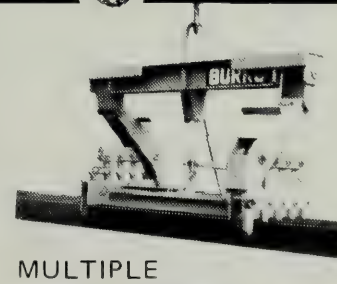

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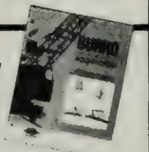

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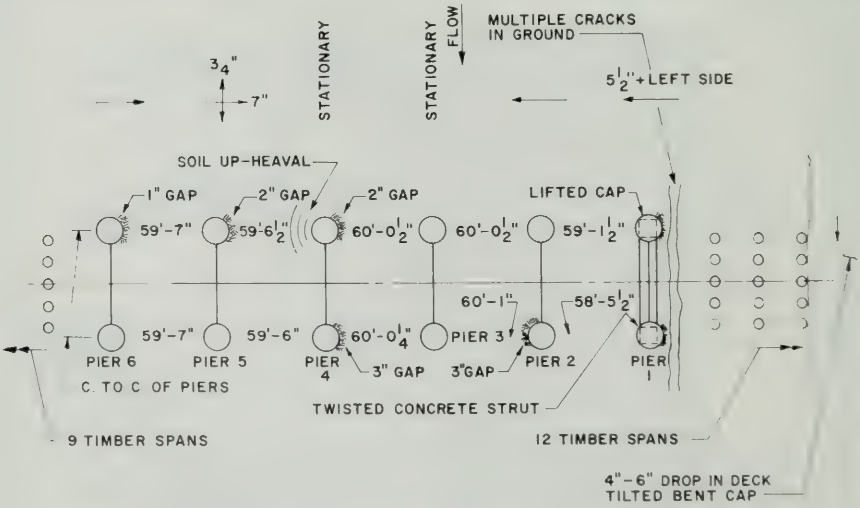
The cylinder piers were all in good condition, except Pier 1, where the concrete beam between cylinders was twisted, tilted and pulled up from the top of one cylinder. Most of the cylinders exhibited signs of movement during the quake, as noted by soil heaval adjacent to the cylinders, which left a gap between the soil and the cylinders. Relative movement of the piers is shown in the sketch. Piers 3 and 4 appear to have remained stationary.

The ends of the five girder spans were in contact with each other, but otherwise had not structural damage. Other than repairing Pier 1, only several bearing shoes will require repositioning to place the girder bearing stiffeners over the bearings. To expedite opening the bridge to traffic, a temporary pile bent was driven on either side of Pier 1 to support the girder span and the stringers of the timber trestle.

A timber pile cap of the trestle had also rotated due to thrust from the timber stringers.



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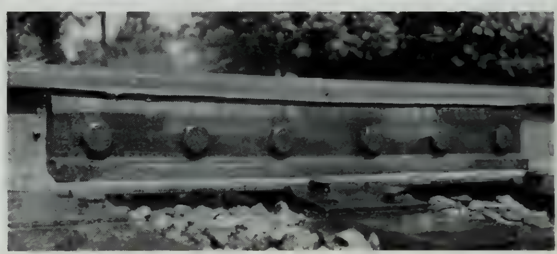

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REPORT OF SUBCOMMITTEE C-2-89
Subcommittee Chairman: R. P. Howell
Vice Chairman: P. L. Shaw

A SURVEY OF WHEEL-ON-RAIL TRACTION VEHICLES CAPABLE OF RUNNING AT 200 KM/H OR MORE

INTRODUCTION

This survey of high speed rail vehicles is provided for information purposes only. It reviews the progress of high speed train development over the past ten years and gives details of the locomotives and traction units in use at speeds of 200 km/h (125 mile/hour) or more.

The French TGV-Atlantique is presently the fastest train in service, with routine operation at 300 km/h (186 miles/hour) on the new Paris-Atlantic route. A modified TGV-A train set recently raised its own world railway speed record to 515 km/h (320 miles/hour) during a series of test runs on a newly commissioned section of high speed route.

Review of this survey demonstrates the major progress achieved during the 1980's. It provides the means to understand underlying trends and prospective parameters for future high speed rail projects now in the planning stage.

GENERAL ASPECTS

So much has been published recently about high speed guided ground transportation, sometimes referring to conventional wheel-on-rail systems, sometimes to magnetic levitation, that there is a need to clarify that for the purpose of this survey only the conventional systems are included. Furthermore, in order to limit the number of traction vehicles surveyed, only those capable of running at maximum speeds of about 200 km/h have been considered.

It is recognized that out of the total cost for establishing a high speed rail connection the infrastructure is dominating. The initial cost for the vehicles may be only 7 to 19% of the total investment. However, because of the interaction between vehicles and infrastructure, the design of the vehicle must take into account the main parameters of the infrastructure and vice versa. Since experience with high speed passenger rail transportation is very limited in North America in

Guided transportation — Some recent speed records

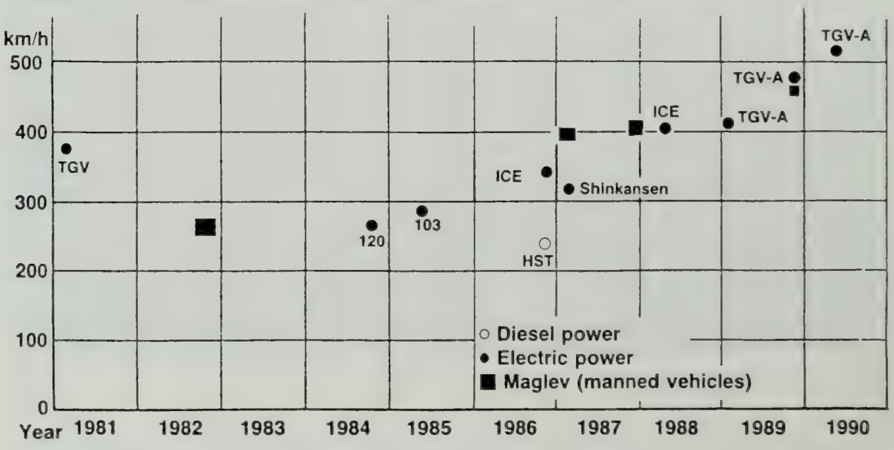

Figure 1

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comparison to the case in Europe or Japan, the scope of this paper is to give an overview of the worldwide development of rail traction vehicles capable of running at 200 km/h or more. Knowledge about what has been achieved may help planners and engineers as well as the railroads themselves in understanding trends and possibilities for future high speed rail.

Figure 1 shows the recent development of rail speed records. It is, however, important to bear in mind the differences between:

(a) the maximum speed attained for a short period of time during a test especially if the vehicle (e.g. its gear ratio or pantograph) and/or the infrastructure (e.g. the catenary) has been changed from standard arrangement in order to perform the test
(b) the maximum speed for which a vehicle has been designed, sometimes in anticipation of further improvements in the infrastructure
(c) the maximum speed intended for revenue service
(d) the average speed attainable in revenue service over a specific line taking into account track-related speed limits and stops at stations

SOME ASPECTS OF HIGH SPEED GROUND TRANSPORTATION

- Use of existing infrastructure or new (dedicated) infrastructure
- Conventional wheel-on-rail concept versus magnetic levitation
- Curve radii and superelevation
- Tilting of power unit(s) and/or passenger coaches?
- Distribution of power(ed wheels) along the train
  - Locomotive-hauled train, power car(s), all or most axles powered
- Propulsion
  - Mechanical, hydraulic or electrical transmission of power
  - Type of traction motor
    - Three-phase (asynchronous or synchronous)
    - Direct current (series or separate excitation)
    - Linear motor
  - Type of control system
- Primary power
  - Non-electrical
    - Diesel engine
    - Turbine
  - Electrical
    - Catenary
      - Single-phase
      - DC
    - Magnetic Levitation
      - Long or short stator
      - Synchronous or asynchronous motor
      - Attractive or repulsive system
    - Fuel cells
- Some mechanical aspects
  - Maximum permissible axle load
  - Wheel diameter and gear ratio
  - Axle arrangement (BoBo, BoBoBo, CoCo)
  - Articulated train?
- Economic aspects
  - Ridership and revenue
  - Cost
  - Financing
  - Possible use for freight transportation

Figure 2

Some aspects of high speed guided ground transportation that have to be considered when planning for new connections, building or updating infrastructure, purchasing or rebuilding traction vehicles, etc., are listed in figure 2. Since the speeds we are talking about here are substantially higher than is the the case for today's relatively low speed freight transportation it should not be surprising that some of the vehicle parameters (e.g. power rating, axle load, wheel base, nose shape, to mention just a few) have to be selected quite differently from what they are for a freight locomotive. Two examples are given in Figures 3 and 4 which show French guidelines for the general design of traction vehicles for high speed rail transportation, the first for existing infrastructure, the second for new infrastructure with alignment suitable for speeds up to 300 km/h.

BODY-TRUCK DYNAMIC CHARACTERISTICS AFFECTING THE GENERAL DESIGN OF A VEHICLE FOR HIGH SPEED RAIL TRANSPORTATION

The case of existing infrastructures

Dynamics of two-axle bogie (stability on straight and curved track)		**Transverse decoupling[1] between body and bogies**	
1 *a*	Maximum axle loading = 16.5/17 t.	1 *b*	Maximum possible symmetry of body mass distribution (longitudinal balance)
2 *a*	Wheelbase not less than 2.600 m.	2 *b*	Frequency of angular hunting of body not more than 1 Hz
3 *a*	Mass of bogie not more than 8/8.4 t	3 *b*	Dynamic decoupling[1] and kinematics of the secondary suspension and the drive to the bogie
4 *a*	$K = \frac{\text{mass of body}}{\text{mass of bogie}} \geqq 2.3/2.4$		

[1] The decoupling of the body from the bogies is also of fundamental importance in the standard of comfort provided by the suspension and the requirements in this context are, moreover, essentially associated with those governing the vertical decoupling and the sway characteristics. The distribution of the body masses, the layout, the suspension damping and the kilometres of the connections between the body and the bogies should all be so defined as to satisfy all these conditions.

French guidelines according to Guy Senac
Source: French Railway Techniques 19(1976)3: pp. 107-143.

Figure 3

The utilization of adhesion is very much lower at high speeds of the order of 200 to 300 km/h than is the case at low speeds. Figure 5 shows measured values on the Shinkansen, indicating an adhesion of only about 4% under wet conditions at 250 km/h.

Before introducing a new or improving an existing high speed rail facility it is, of course, necessary to find out with a high degree of probability if the rate of return on the investment is going to be satisfactory. Besides the cost of the investment, the amount of revenue must be estimated. The ridership depends to a certain extent on traveling time needed which is inversely proportional to the

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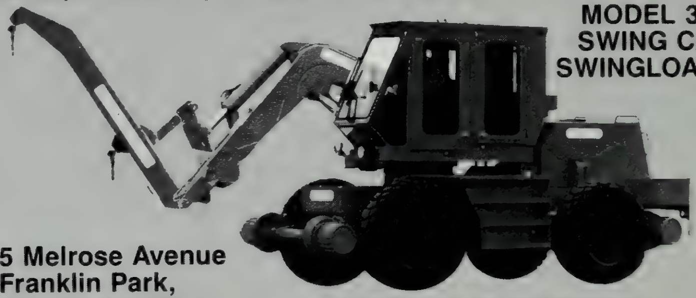

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BODY-TRUCK DYNAMIC CHARACTERISTICS AFFECTING THE GENERAL DESIGN OF A VEHICLE FOR HIGH SPEED RAIL TRANSPORTATION

The case of new infrastructures with alignments to suit up to 300 km/h

Dynamics of two-axle bogie (stability on straight and curved track)[1]		Transverse decoupling[2] between body and bogies	
1 *a*	Maximum axle loading: 16/16.25 t.	1 *b*	Maximum possible symmetry of body mass (longitudinal balance)
2 *a*	Wheelbase: 2.9/3 m	2 *b*	Frequency of angular hunting of body not more than 1 Hz[3]
3 *a*	Mass of bogie not more than 7.5/8 t.[4] with only the transmission components mounted on the power bogie	3 *b*	Dynamic decoupling and kinematics of the secondary suspension of the bogie drive and of the hunting-control device
4 *a*	$K = \frac{\text{mass of body}}{\text{mass of bogie}} \geq 2.5$		

[1] Curves of radius not less than 3200 m. negotiable at 260 km/hr. with a maximum cant deficiency of 130 mm

[2] See note (1) to table II.

[3] Dependent upon the distance between bogie pivot centres, in addition to the radius of gyration of the body (distribution of masses) and the transverse flexibility characteristics of the secondary suspension.

[4] Reduction of the mass of the bogie enables the wheel-base to be reduced (to 2.600 m. for example, if the mass can be reduced to 6 t.)

Note: The development of the TGV stock illustrates the application of items 2 *a* to 4 *a*. By replacing the Y 225 bogies of train TGV 001, which still have a mass of 10.2 t with a wheel-base of 2.600 m. by bogies in accordance with these items (cf *fig.* 13 and 14), the hunting control torque necessary at 260 km/hr. will be reduced and will even be zero immediately after re-profiling.

French guidelines according to Guy Senac
Source: French Railway Techniques 19(1976)3: pp. 107-143.

Figure 4

average speed and at least approximately inversely proportional also to the maximum train speed. Investigations in Britain, France, Germany and Italy are summarized in Figure 6 showing in two bands the estimated rail share of passenger traffic as a function of maximum train speed. The upper band relates to domestic traffic (usually up to a distance of about 500 km), while the lower band corresponds to international traffic in Europe over somewhat longer distances. It is evident that high speed rail can be competitive with the airlines if maximum speeds of, say, 300 km/h can be attained and the traveling distance is not more than about 600 km.

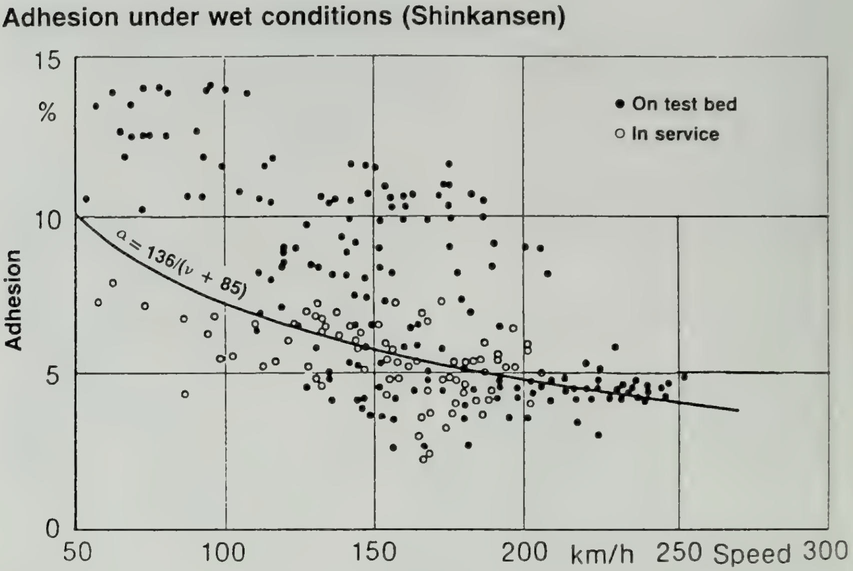
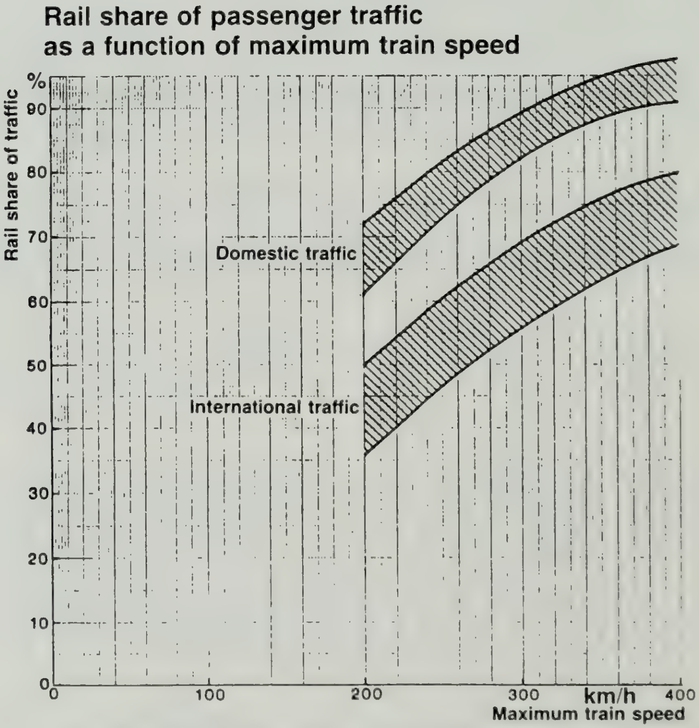


Figure 5



Investigations in Britain, France, Germany and Italy

Source: Weber, Internationales Verkehrswesen 34(1982)2: p. 92.

Figure 6

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Before the surveyed traction vehicles are described in some detail it might be useful to emphasize that a number of high speed train configurations are possible, the principal ones shown in Figure 7 where filled circles indicate powered axles:

1. Coaches hauled by a single locomotive is the classical arrangement and still in widespread use
2. If the rating of one locomotive is not sufficient, the coaches may be hauled by two locomotives. A typical example is the "Russian Troika" between Moscow and Leningrad
3. Becoming more and more popular is an arrangement with one power car at each end of the train with a number of intermediate passenger coaches
4. All (or practically all) axles can be powered in an EMU train
5. The precursor of the TGV trains of today was a fully articulated train TGV 001 in which all axles were powered
6. The first commercial TGV train between Paris and Lyon was semi-articulated with six axles powered at each end of the train (Class 23000)
7. The new TGV Atlantique is also semi-articulated but only four axles at each end of the train are powered (Class 24000)
8. The multiple-end unit train ER 200 in the USSR has at each end of the train a non-powered car with a driver's cab, but all the intermediate cars are powered
9. The Italian "Pendolino" train has only the inner axles of each truck powered.

In the description of traction vehicles that follow, reference will be made to these principal train configurations.

There are a number of classifications possible for subdividing this information material, e.g. according to non-electrified railroads versus electrified railroads; or a classification related to locomotives, power cars, or multiple-unit trains; or all traction vehicles listed strictly in chronological order as they have emerged. There may be certain advantages and disadvantages with all these classifications. Arbitrarily selected for this paper is a listing of the traction vehicles according to the country in which they are primarily used, and the countries are listed in a sequence related to the chronological order in which they started utilizing trains for 200 km/h or more.

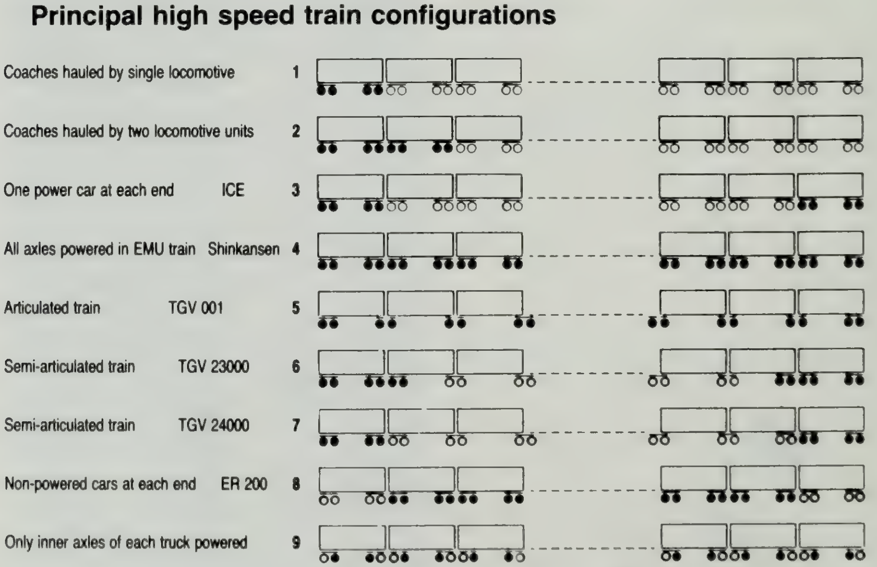
Principal high speed train configurations
Coaches hauled by single locomotive 1
Coaches hauled by two locomotive units 2
One power car at each end ICE 3
All axles powered in EMU train Shinkansen 4
Articulated train TGV 001 5
Semi-articulated train TGV 23000 6
Semi-articulated train TGV 24000 7
Non-powered cars at each end ER 200 8
Only inner axles of each truck powered 9

Figure 7

HIGH SPEED TRACTION VEHICLES

FRANCE

Early high-speed electric locomotives (all train configuration 1, Fig. 7)

After the end of World War II, the development of traction vehicles for high speed was not limited to but in many respects led by the French National Railways (SNCF) and the French railroad manufacturing industry. In the early 1960's SNCF had already available 13 electric locomotives capable of maximum speeds in the range of 200 to 250 km/h. Many of these, and some others with modifications, were used for high speed tests to further extend the knowledge about what actually happens in a railroad system when trains are run at even higher speeds.

On March 28 and 29, 1955, two electric locomotives, CC 7107 and BB 9004, both modified for the tests (Figures 8 and 9) attained a maximum speed of 331 km/h. This was a world speed record for rail which was not beaten until 1981.

Electric locomotive CC 7107 modified for high speed tests (France)
Direct current traction motors

Maximum speed attained	331 km/h	Tractive effort at start	
Power at rail		Wheel diameter (new)	1250 mm
Power supply	1500 volts DC	Special gear ratio	
Weight of locomotive	107 tonnes	Wheel base in truck	2335 + 2510 mm = 4845 mm
Axle load	17.8 tonnes		

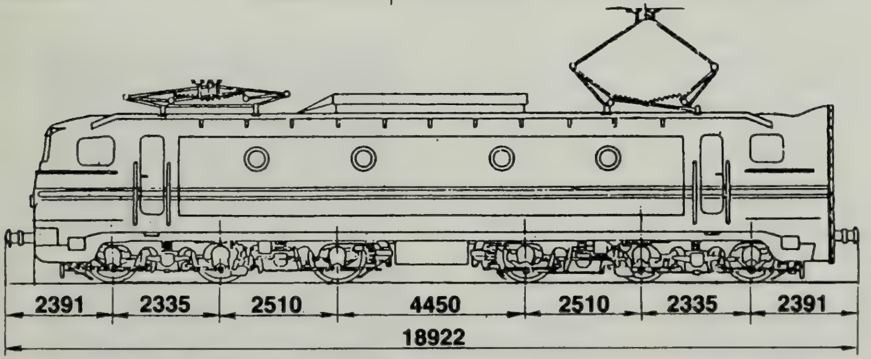

Figure 8

The fastest standard locomotives in France at the time were officially the two Bo'Bo' locomotives 9291 and 9292, geared for a maximum speed of 250 km/h (Figure 10). Almost as fast were the four 6-axle, 4-system locomotives of Class 41000 (Figure 11) which in high gear would run at a maximum speed of 240 km/h.

At the end of World War II only 3530 route-km of the French railroads were electrified but electrification was extended by 553 km in 1958 only. Electrification opened the possibilities to increase substantially the power ratings of the traction vehicles, but in France interest increased also in using gas turbines for high speed trains on non-electrified lines. This development was run in parallel to improving the electric traction vehicles even further.

Class RTG turbine-hydraulic trains (Train configuation 3, Fig. 7)

After successful tests with various types of turbine-driven vehicles in which the TGS experimental turbine train reached 252 km/h on October 19, 1971, a commercial turbine-hydraulic train Class RTG

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Electric locomotive BB 9004 modified for high speed tests (France)

Direct current traction motors

Maximum speed attained	331 km/h	Wheel diameter (new)	1250 mm
Power at rail		Special gear ratio	
Power supply	1500 volts DC	Wheel base in truck	3200 mm
Weight of locomotive	83 tonnes		
Axle load	20.8 tonnes		
Tractive effort at start			

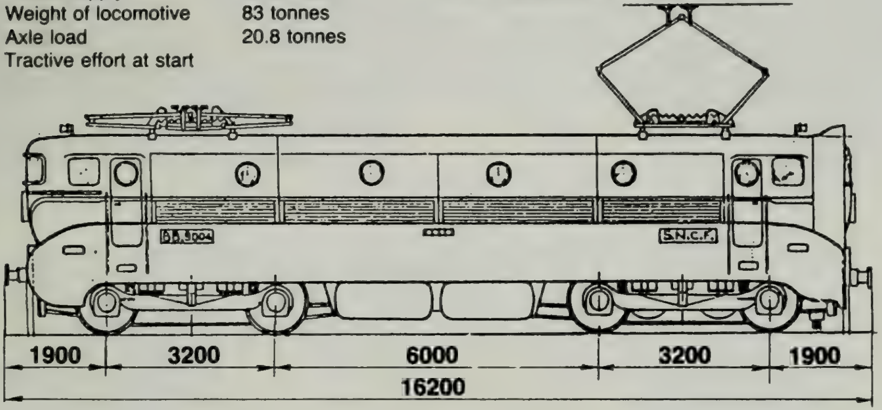

Figure 9

Electric locomotives 9291 and 9292 in France

Maximum speed	250 km/h	Axle load	21 tonnes
Power at rail	4240 kW continuously	Tractive effort at start	147 kN
Power supply	1500 volts DC	Wheel diameter (new)	1250 mm
Weight	84 tonnes	Wheel base in truck	3200 mm

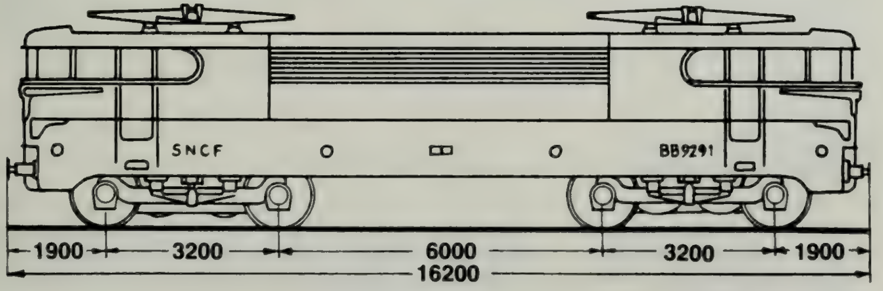

Figure 10

Electric 4-system locomotive Class 40100 for France
Direct current traction motors

Maximum speed	240 km/h
Power at rail	4480 kW continuously (for second series)
Power supply	25 kV, 50 Hz, and 15 kV, 16 2/3 Hz 3000 volts DC and 1500 volts DC
Weight	107 tonnes
Maximum axle load	17.8 tonnes
Tractive effort at start	147 kN
Wheel diameter (new)	1100 mm
Gear ratio	1:1.221
Wheel base in truck	1608 + 1608 mm = 3216 mm

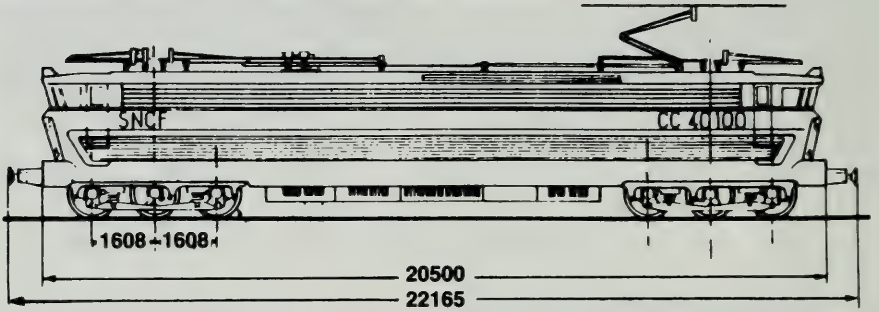

Figure 11

was developed. The first train was delivered to SNCF on December 1, 1972. The main parameters are shown in Figure 12.

Amtrak leased two RTG trains of which the first arrived in the United States in August 1973. Rohr Industries was in 1973 granted a license by the French company ANF to build these turbine-powered trains in the USA. Several trains of an Americanized version, the RTL, have been delivered to Amtrak, and the RTG has also been sold to Iran and Egypt.

With further extended electrification in France, some turbotrains there will gradually be withdrawn from service.

Turbine-electric train TGV 001 (Train configuration 5, Fig. 7)

In July 1969, SNCF ordered an experimental gas turbine train capable to run on tests up to 300 km/h. Compared with the previous gas turbine trains, the TGV 001 received a much better streamlining and an electric transmission. The train was wholly articulated, made up of five bodies mounted in six trucks. The two end cars incorporated a driving cab and housed the power pack. The articulated arrangement allowed the floor to be lowered, and the total height above rail level was reduced to only 3.4 meters. The center of gravity was lowered by 300 mm, assisting stability. This was said to practically eliminate the need for body-tilting. All axles were powered.

TGV 001 left the manufacturer's works on October 25, 1971, and tests on SNCF track started in April, 1972. This train was the first example of rolling stock designed from the start for speeds in excess of 250 km/h. According to Bernard 1973: "Locomotive-hauled trains, the traditional arrangement inherited from the days of steam traction, are no longer suitable for high speed running."

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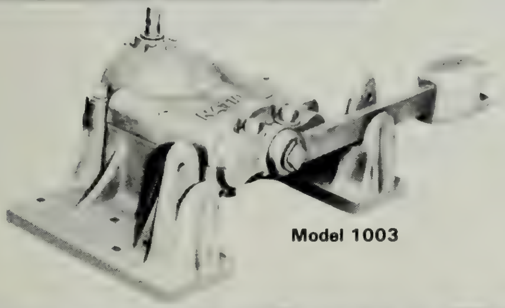
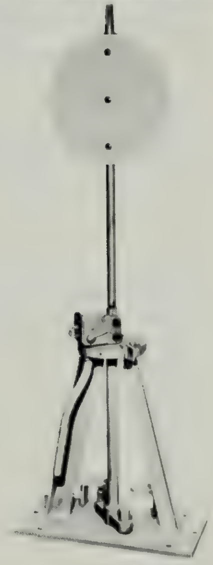


Model 1003

HIGH COLUMN



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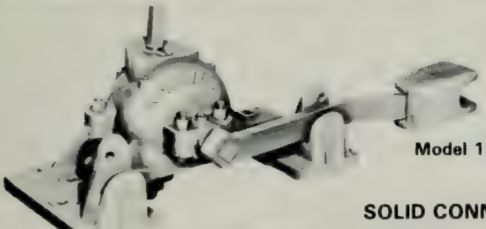

Model 1004

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JAW END

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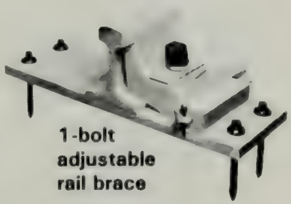

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Turbo-hydraulic power car for the RTG turbotrain in France

Maximum speed	200 km/h
Power at rail	1513 kW for two power cars
Rating of one main turbine	1100 kW at 5700 r/min (Turmo X)
Weight of one power car	60 tonnes
Maximum axle load	16 tonnes
Tractive effort at start	133 kN for two power cars
Wheel diameter (new)	860 mm
Wheel base in truck	2700 mm

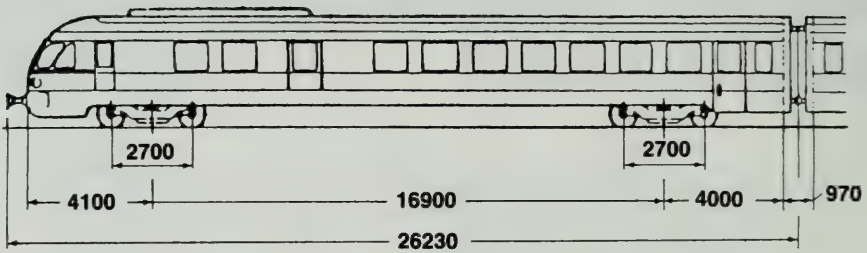

Figure 12

Turbo-electric power car TGV 001 in France
Direct current traction motors

Maximum speed	270 km/h
Power at rail	3427 kW continuously for complete train
Rating of one main turbine	1100 kW at 5785 r/min (Turmo X)
Weight of one power car	48.7 tonnes
Maximum axle load	16.8 tonnes
Tractive effort at start	167 kN for complete train
Wheel diameter (new)	900 mm
Wheel base in truck	2600 mm

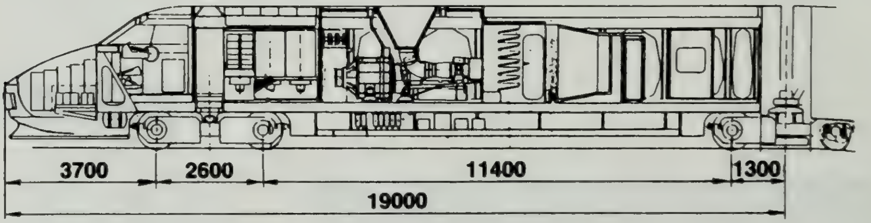

Figure 13

The train was used for aerodynamic studies up to 300 km/h, and on December 8, 1972, it attained 318 km/h. It's principal parameters are shown in Figure 13.

Experimental electric motor coach Z 7001

To get information about the behavior of an electric traction vehicle at speeds up to 300 km/h, SNCF converted an electric power car to test electric equipment and trucks likely to be used for a straight-electric version of the TGV 001 turbotrain.

The body of car Z 7115 was fitted with more streamlined ends and mounted on the new Y 226 trucks designed for high speed and having a wheel base of 2900 mm. The vehicle was renamed Z 7001 (Figure 14). A carden shaft transmission allowed the same traction motors as used on the TGV 001 to be mounted under the body and not in the trucks. Current collection from the catenary presented no problem even at 300 km/h.

Experimental electric motor coach Z 7001 in France
Direct current traction motors

Maximum speed	300 km/h	Wheel diameter (new)	910 mm (880 mm half-worn)
Power at rail	2050 kW continuously		
Power supply	1500 volts DC	Gear ratio	28:45 ≈ 1:1.607
Maximum axle load	17.5 tonnes	Wheel base in truck	2900 mm
Tractive effort at start	31.3 kN	Weight of one car	70 tonnes

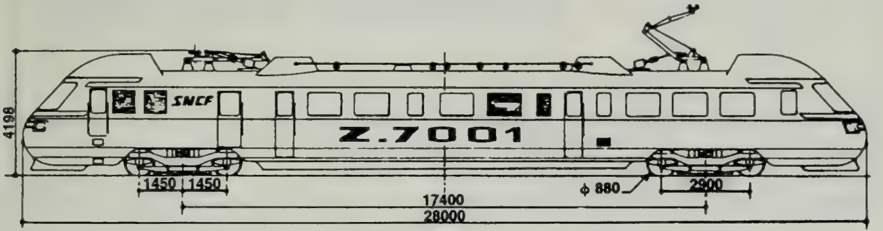

Figure 14

Power car Class 23000 (Train configuration 6, Fig. 7)

Class 23000 is the high speed train for the dedicated Paris-Lyon line (TGV SudEst). Contracts were awarded on February 12, 1976, for design and construction of two pre-production train sets and later for a total production of 109 sets. The first train set was delivered on July 28, 1979.

A Class 23000 train attained 260 km/h on December 18, 1980, and 380 km/h on February 26, 1981. Revenue service started on September 27, 1981. The principal parameters for Class 23000 are shown in Figure 15.

Electric dual system end power car Class 23000 (TGV SudEst)
Direct current traction motors

Maximum speed	270 km/h
Power at rail	6300 kW on 25 kV, for complete train
Power supply	25 kV, 50 Hz, or 1500 volts DC
Weight of one end power car	
Maximum axle load	<17 tonnes
Tractive effort at start	215 kN for complete train
Wheel diameter (new)	920 mm
Wheel base in truck	3000 mm

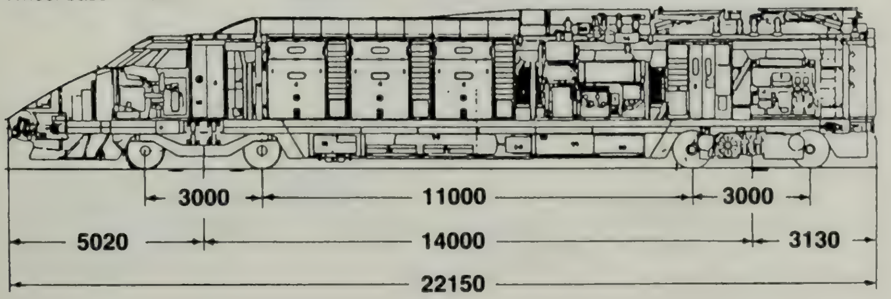

Figure 15

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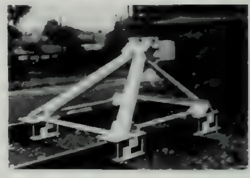

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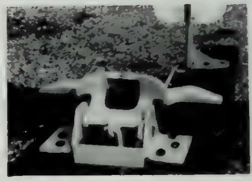

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Power car Class 24000 (Train configuration 7, Fig. 7)

Thoughts about a second TGV line (from Paris mainly to the Southwest) emerged in 1977. On April 15, 1982, it was decided that the new line, TGV Atlantique, should be designed for 300 km/h and that each train should consist of 12 cars. The French government gave it's formal backing on November 4, 1982. Work on the new line started on January 7, 1985.

Two power cars of Class 23000, numbers 23176 and 23177, were modified to use three-phase synchronous motors and renamed 02301 and 02302.

Contracts for a total of 95 train sets Class 24000 were signed on November 18, 1985. The first set was publicly demonstrated on April 14, 1988. The cost per train was said to be 73 million French Francs. The first train entered service in existing timetables on January 23, 1989. Tracklaying on the new line was completed on February 2, 1989. Class 24000 has attained 290 km/h in February 1986, 356 km/h in September 1986, 410 km/h in January 1989, 482 km/h in December 1989 and 515 km/h (320 miles/hour) in May 1990.

Figures 16 and 17 give more information about Class 24000.

Electric dual-system power car Class 24000 (TGV Atlantique)
Three-phase synchronous traction motors

Maximum speed	300 km/h	Maximum axle load	17 tonnes
Power at rail	8800 kW (for two power cars)	Tractive effort at start	210 kN (for two power cars)
Power supply	25 kV, 50 Hz, or 1500 volts dc		
Weight of one power car	68 tonnes	Wheel diameter (new)	920 mm

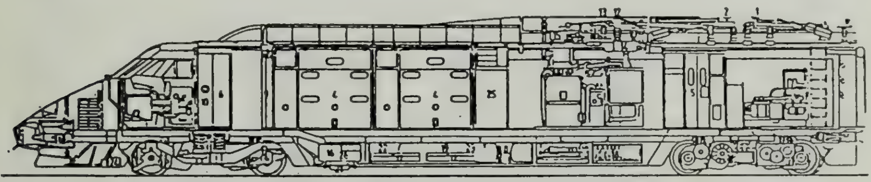

Figure 16

Power car TGV 24000 (Atlantique)
Maximum service speed 300 km/h Three phase synchronous traction motors

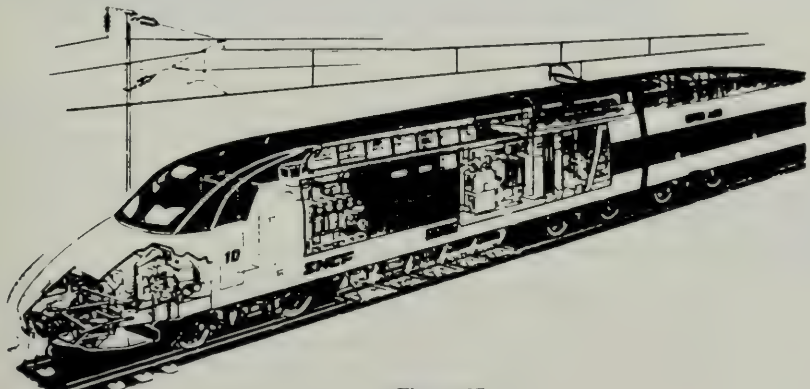

Figure 17

TAV trains for Spain (Train configuration 7, Fig. 7)

An order for 24 TAV trains was announced by RENFE on December 23, 1988. These trains will be very similar to the Class 24000 in France. The cost per train was 2.14 billion pesetas.

French prototype locomotives with three-phase traction motors (Train configuration 1, Fig. 7)

SNCF asked in 1979 two groups of electrical suppliers to each equip one locomotive of Class 15000 with three-phase traction motors.

Locomotive BB 15055 was equipped with a pair of three-phase synchronous traction monomotors. MTE and Jeumont-Scheider carried out the conversion and the locomotive was renamed BB 10004. Its principal parameters are shown in Figure 18. Running trials began in February 1982.

Electric prototype locomotive BB 10004 in France

Three-phase synchronous traction motors

Maximum speed	200 km/h	Tractive effort at start	320 kN
Power at rail	5600 kW continuously	Wheel diameter (new)	1250 mm
Power supply	25kV, 50 Hz	Wheel base in truck	2800 mm
Weight of locomotive	88 tonnes	Gear ratio	33:73≈
Axle load	22 tonnes		≈1:2.212

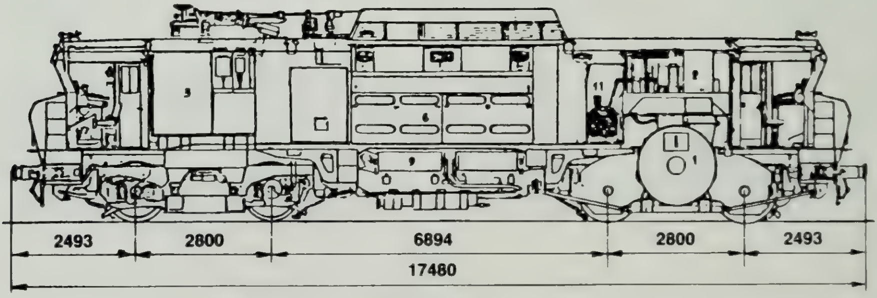

Figure 18

Another locomotive, originally BB 15007, later BB 7003, was placed on new trucks, each equipped with two three-phase asynchronous traction motors. This conversion was done by Alsthom and Francorail. The locomotive was renamed BB 10003, and some of its parameters are shown in Figure 19. After tests at Alsthom in October 1985, road tests started on January 28, 1985.

Successful tests on the BB 10004 resulted in SNCF placing an order for two pre-production locomotives, BB 20011 and BB 20012 for dual-system service on 25 kV, 50 Hz, as well as on 1500 volts DC. Some details are shown in Figure 20. The first of these locomotives began running trials on June 24, 1985.

Electric dual-system locomotives Class 26000 (Train configuration 1, Fig. 7)

SNCF placed in July 1984 an order for 44 locomotives Class 26000 with electrical equipment based on the successful tests with the prototype locomotives 10004, 20011 and 20012. The locomotives of Class 26000 are also to be known by the acronym Sybic (Synchrone-bicourant). They are intended both for hauling freight trains at up to 160 km/h and passenger trains with a maximum speed of 200 km/h. The principal features of the Class 26000 locomotive are shown in Figure 21.

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Electric prototype locomotive BB 10003 in France

Three-phase asynchronous traction motors

Maximum speed	160 km/h	Maximum axle load	23 tonnes
Power at rail	5600 kW	Tractive effort at start	355 kN
Power supply	25 kV, 50 Hz	Wheel diameter (new)	1150 mm
Weight of locomotive	92 tonnes	Wheel base in truck	2750 mm

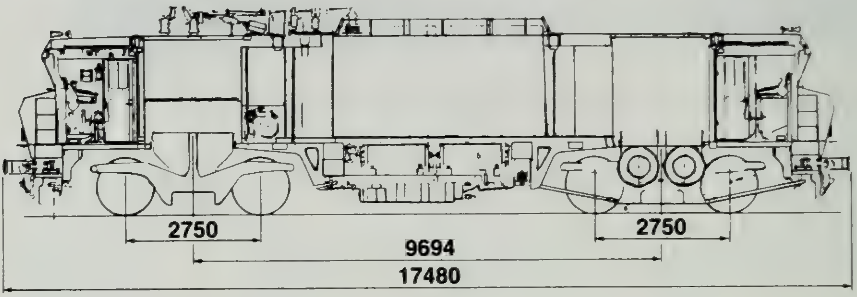


Figure 19

Electric prototype locomotives BB 20011 and 20012 in France

Three-phase synchronous traction motors

Maximum speed	200 km/h	Maximum axle load	22.7 tonnes
Power at rail	5600 kW	Tractive effort at start	320 kN
Power supply	25 kV, 50 Hz, and 1500 volts DC	Wheel diameter (new)	1250 mm
		Wheel base in truck	2800 mm
Weight of locomotive	90.5 tonnes		

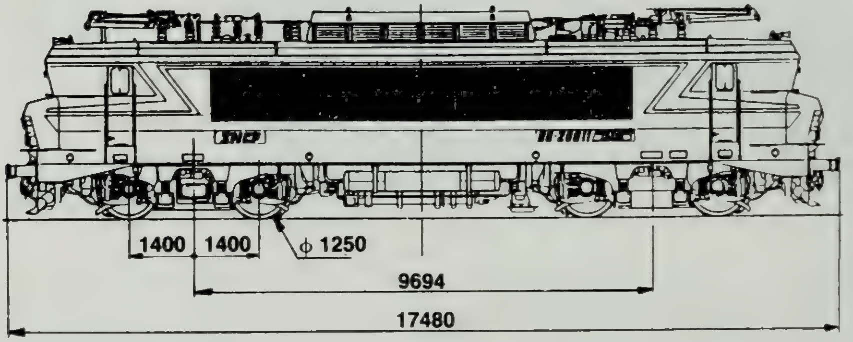


Figure 20

The Dutch Railways completed early in 1988 a study showing that the French synchronous motor concept used in Class 26000 would be best suited for the future Dutch Class 1700 locomotives which from 1994 will take over the duties of 86 older locomotives.

Electric dual-system locomotive Class 26000 (Sybic) (France)

Three-phase synchronous traction motors

Maximum speed	200 km/h	Weight of locomotive	90 tonnes
Continuous power	5600 kW	Tractive effort at start	320 kN
Power supply	25 kV, 50 Hz, or 1500 volts DC		

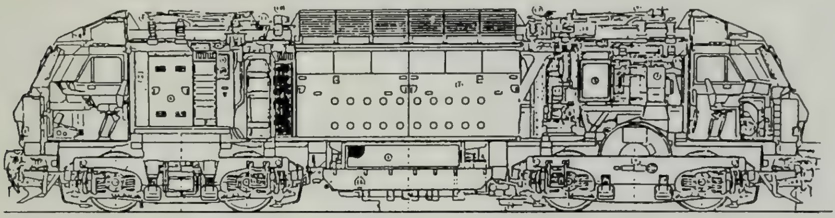


Figure 21

JAPAN

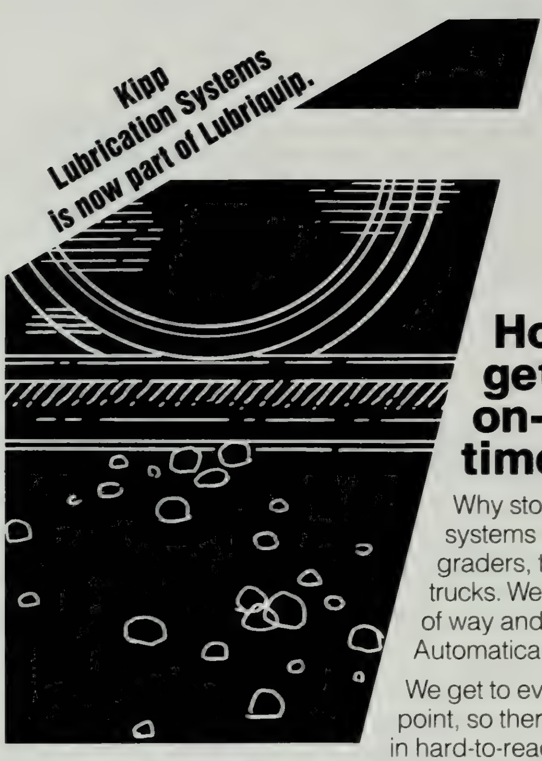
The Shinkansen multiple-unit trains (Train configuration 4, Fig. 7)

Tokaido was the name of the most important road in feudalistic Japan, connecting Edo (now Tokyo) with Kyo (now Kyoto). After the restoration in 1868, narrow gauge (1067 mm) railroads were built throughout Japan, and the one linking Tokyo with Kyoto was the most important, opened in 1891. In 1913 this line was double-tracked. After the end of World War II railroad electrification was introduced in Japan and again priority was given to the Tokaido line. Its electrification was completed in 1956. However, it soon became evident that the line was reaching saturation, and it was decided to build a New Tokaido Line (NTL), this time using standard gauge (1435 mm). "New line" in Japanese is "Shinkansen," and this one became known internationally as the Tokaido Shinkansen. It was opened on October 1, 1964, in time for the 18th Olympic Games held in Tokyo.

After a comparison had been made between various possible systems, the Tokaido Shinkansen was built with a 25 kV, 60 Hz, catenary, and it was decided that the motive power should be distributed along the whole length of the train; in other words, electric multiple unit trains were to be used. After a prototype train attained 257 km/h on a test section of the new line in March 1963, an order was placed for 180 two-car groups (now called the Shinkansen O Series). These cars are using 185 kW DC traction motors, one per axle, four motors permanently connected in series. The utilization of adhesion is very low at high speeds, only about 4% at 210 km/h. The motors are frame-mounted and drive the axles through a hollow cardan shaft.

The Shinkansen Series O cars (Figure 22) have a maximum speed of 210 km/h, a maximum axle load of 16 tonnes and a wheel base in truck of 2500 mm.

The Tokaido line between Tokyo and Shin-Osaka has a length of 515 km. On March 15, 1972, the first part of the Sanyo Shinkansen between Osaka and Okayamo (162 km) was opened, later extended to Hakata so that the total length now is 554 km. In 1982, two lines were opened, the Tohoku Shinkansen (465 km) and the Joetsu Shinkansen (270 km). On both these lines, Shinkansen Series 200



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trains (Figure 23) are used with DC traction motors rated 230 kW. The wheel base in truck is the same, 2500 mm, but the axle load is somewhat higher, 17 tonnes, in comparison to Series O.

Another type of train, Shinkansen Series 100 (Figure 24), was ordered in 1984 and the first train was delivered in March 1985. It is also powered by 230 kW DC traction motors using the same wheel base in truck, 2500 mm, but the axle load has been reduced to 15 tonnes.

End car of Shinkansen train Series O

Direct current traction motors

Maximum speed	210 km/h	Maximum axle load	16 tonnes
Power at rail	740 kW per car (continuously)	Tractive effort at start	18.9 kN per car
		Wheel diameter (new)	910 mm
Power supply	25 kV, 60 Hz	Gear ratio	29:63 ≈ 1:2.17
Weight of one car	64 tonnes	Wheel base in truck	2500 mm

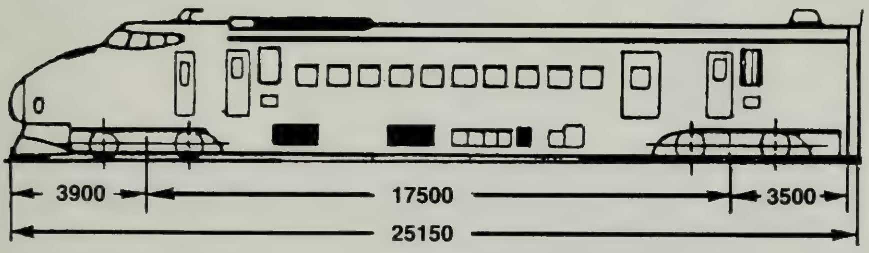

Figure 22

End car of Shinkansen train Series 200

Direct current traction motors

Maximum speed	210 km/h	Maximum axle load	17 tonnes
Power at rail	920 kW continuously per car	Tractive effort at start	30.4 kN per car
		Wheel diameter (new)	910 mm
Power supply	25 kV, 50 Hz	Gear ratio	29:63 ≈ 1:2.172
Weight of one car	68 tonnes	Wheel base in truck	2500 mm

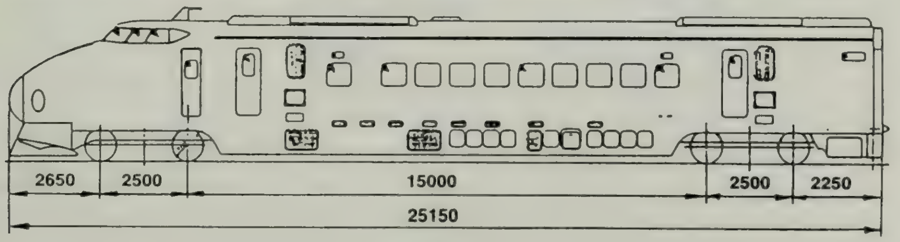

Figure 23

End car of Shinkansen train Series 100
Direct current traction motors

Maximum speed	210 km/h
Power at rail	920 kW continuously per powered car
Power supply	25 kV, 60 Hz
Weight of one car	60 tonnes

Maximum axle load	15 tonnes
Tractive effort at start	40 kN per powered car
Wheel diameter (new)	910 mm
Gear ratio	27:65 ≈ 1:2.407
Wheel base in truck	2500 mm

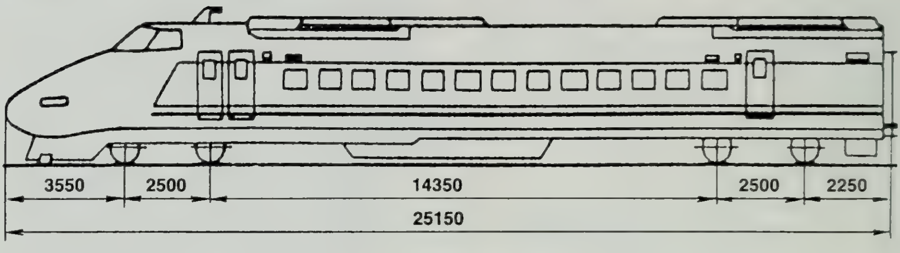

Figure 24

Since 1986, the Japanese are studying the possibilities of introducing a Shinkansen Series 300 train (Figure 25) which will be powered by three-phase asynchronous traction motors. Since these are more compact than DC traction motors, clearance between rail head and vehicle floor can be reduced by 150 mm, and height of the cars is planned to be 3600 mm instead of 4000 mm. One proposal is to use six powered cars in the middle of the train with one non-powered car with driver's cab at each end of the train. Axle load may be limited to 14 tonnes. Traction motors will be rated 320 kW.

Powered car of Shinkansen train Series 300
Three-phase asynchronous traction motors

Maximum speed	300 km/h
Power at rail	1280 kW continuously per power car
Power supply	25 kV, 50 or 60 Hz
Weight of one car	56 tonnes (design target)

Maximum axle load	14 tonnes (design target)
Tractive effort at start	41.5 kN per powered car
Wheel diameter (new)	
Gear ratio	
Wheel base in truck	

No outline of vehicle available.

Figure 25

In November 1986, Series O trains were allowed a maximum speed of 220 km/h. Series 100 should be able to run at 260 km/h on the Sanyo line and 230 km/h on the Tokaido line.

In January 1988, the Shinkansen network had a total length of 1834 km.

Performance curves for some of the Shinkansen trains are compared in Figure 26. The improvement achieved with the later versions is partly due to a reduction in the cross-section of the trains (Figure 27) but in the case of Series 300 also to the introduction of three-phase traction motors instead of DC traction motors.

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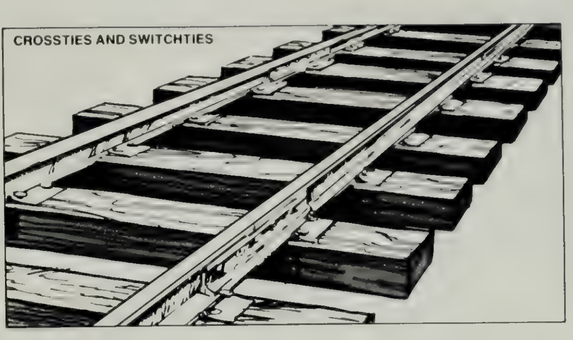

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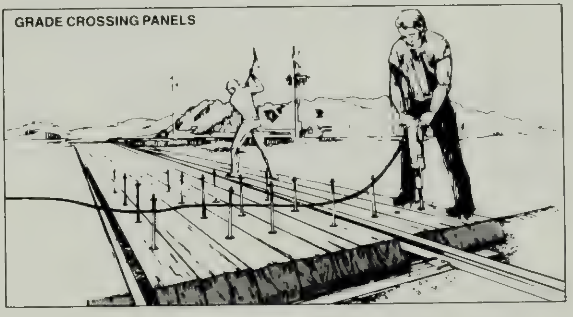

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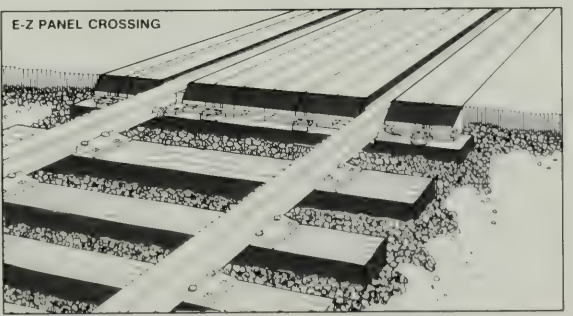

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Performance of various Shinkansen train sets

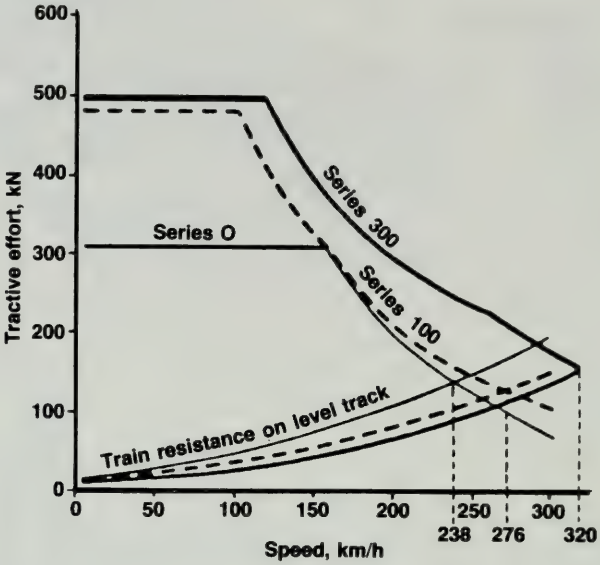

Figure 26

Profiles of various Shinkansen trains
For comparison: the original TGV (Paris-SudEst)

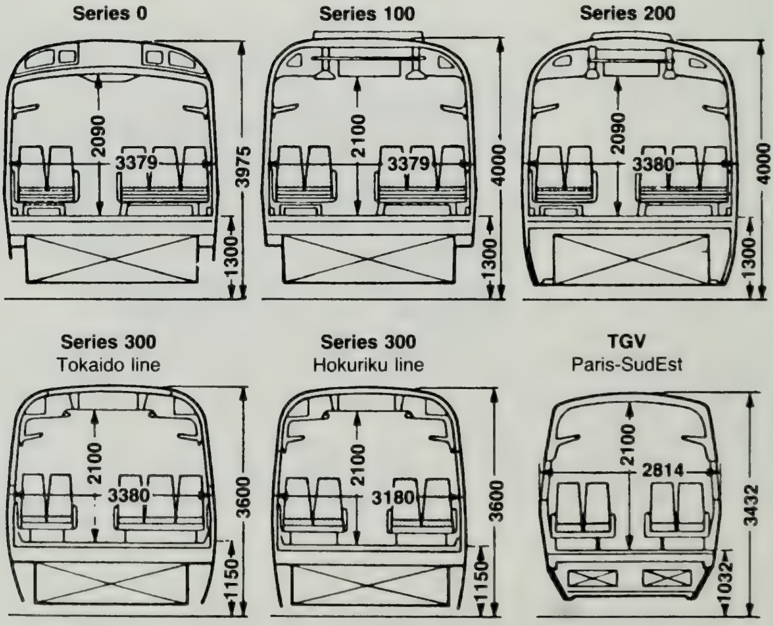

Figure 27

ITALY

Electric motor coach Class ALe 601 (Train configuration 1 or 4, Fig. 7)

The Italian State Railways (FS) was the first European railroad administration to introduce high speed trains composed of motor coaches and trailers. Already in 1937 a Class 200 train attained a maximum speed of 201 km/h. In 1958, the FS commenced the design of a new group of motor coaches (ALe 601) and related trailers. They entered service in 1961 and were then designed for a maximum speed of 180 km/H. During a test on December 5, 1963, coach ALe 601.018 attained a speed of 225 km/h, and at another test in October 1968 an ALe 601 reached 250 km/h. In 1973, the first train of a new improved series of 21 motor coaches ALe 601 entered service (Figure 28). New traction motors made it possible to increase the maximum speed in service to 200 km/h.

Electric motor coach Class ALe 601 in Italy
Direct current traction motors

Maximum speed	200 km/h	Weight of one car	67 tonnes
Power at rail	860 kW continuously	Maximum axle load	17 tonnes
Power supply	3000 volts DC	Tractive effort at start	71 kN
Axle arrangement	Bo'Bo'	Wheel diameter (new)	1040 mm

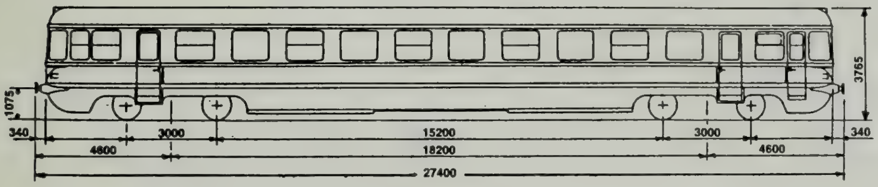

Figure 28

Electric locomotive Class E 444 (Train configuration 1, Fig. 7)

FS ordered four prototype locomotives E 444 in the mid-1960's, geared for a maximum speed of 180 km/h. The first of these locomotives was presented to the media on November 8, 1967, and then

Electric locomotive Class E 444 in Italy
Direct current traction motors

Maximum speed	200 km/h	Tractive effort at start	245 kN
Power at rail	4500 kW cont	Wheel diameter (new)	1250 mm
Power supply	3000 volts DC	Gear ratio	40:77 = 1:1.925
Weight	84 tonnes	Wheel base in truck	2600 mm
Maximum axle load	21 tonnes		

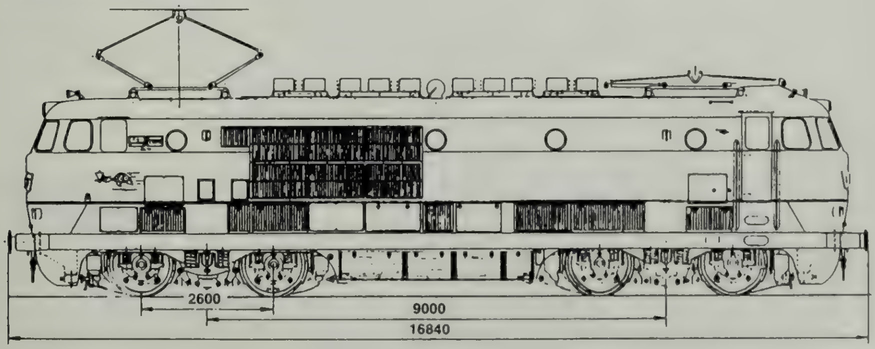

Figure 29

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immediately tested at 210 km/h. A fifth locomotive was provided with chopper control and the gear ratio was changed to allow for a maximum speed of 200 km/h. The power at rail was increased to 4500 kW continuously. Some further details are shown in Figure 29.

Electric locomotive Class E 666 (Train configuration 1, Fig. 7)

Class E 666 is a six-axle version of the E 444. It was the first Italian CoCo locomotive, and a prototype was completed in 1973. The maximum speed is 200 km/h, and Figure 30 shows further parameters of the locomotive.

Electric locomotive Class E 666 for Italy

Direct current traction motors

Maximum speed	200 km/h	Axle load	19.4 tonnes
Power at rail	6030 kW continuously	Tractive effort at start	
Power supply	3000 volts DC	Wheel diameter (new)	1250 mm
Weight of locomotive	116 tonnes	Wheel base in truck	2300+ 2300 = 4600mm

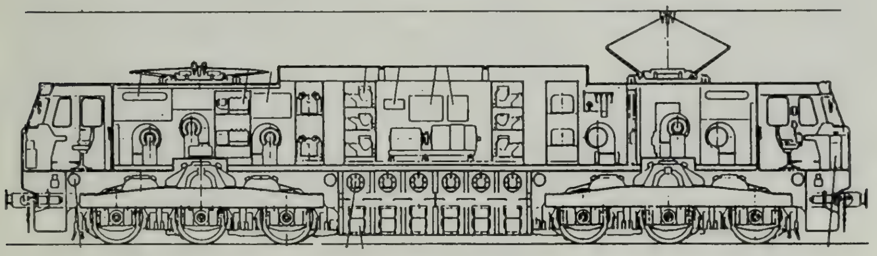

Figure 30

Electric locomotive Class E 402 (Train configuration 1, Fig. 7)

The E 402 is intended as a replacement for the E 444 locomotives. Five locomotives E 402 were ordered from Ansaldo as the first Italian locomotive with three-phase traction motors. Intended maximum speed is 220 km/h but service speed is limited to 200 km/h. The first locomotive attained 230 km/h in 1986. Principal data for the E 402 are shown in Figure 31.

Electric locomotive Class E 402 for Italy

Three-phase asynchronous traction motors

Maximum speed	220 km/h (200 km/h	Axle load	20.5 tonnes
	service speed)	Tractive effort at start	222 kN
Power at rail	5600 kW continuously	Wheel diameter (new)	
Power supply	3000 volts DC	Wheel base in truck	
Weight of locomotive	82 tonnes		

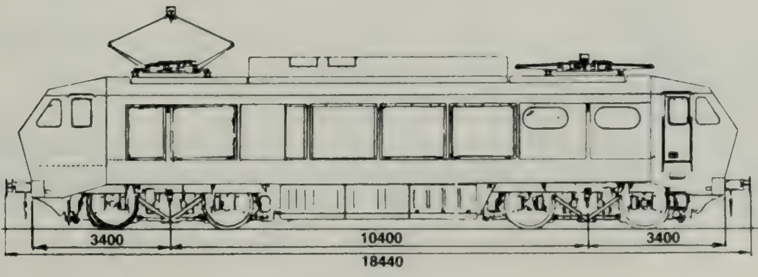

Figure 31

Electric tilting train Class ETR 401 (Train configuration 9, Fig. 7)

FS placed contracts in mid-1970 for a prototype four-coach tilting-body electric train intended for a maximum speed of 250 km/h. Each of the four coaches has two similar trucks with DC traction motors slung beneath the body and driving only the inner axles of the trucks. The train is provided with an electro-hydraulic body-tilting mechanism controlled by signals from an accelerator and a tachometer. The coach bodies are tilted automatically and gradually in response to lateral forces and the velocity of the train. A non-tilting framework is mounted on one track of each pair of coaches to carry a pantograph which will thus retain its normal position with regard to the contact wire of the catenary even when the coach bodies are tilting.

A single unit called Y 160 was demonstrated in March 1972. A complete train ETR 401 started commercial trials on May 30, 1976, and entered public service on July 2, 1976. During a test on May 23, 1985, the ETR 401 attained 255 km/h.

Fiat in Italy and the Spanish railroads RENFE signed in 1972 an agreement for development and construction of a tilting train to be built in Spain.

There is interest both in Germany and Austria for modified types of the ETR 401.

Figure 32 shows some details of the Italian ETR 401. The maximum axle load is very low, only 12.6 tonnes.

Electric tilting coach for Class ETR 401 trains in Italy ("Pendolino")

Direct current traction motors

Maximum speed	250 km/h
Power at rail	440 kW per coach continuously
Power supply	3000 volts DC
Axle arrangement	(1A)'(A1)'
Weight of one coach with full load	49.5 tonnes
Maximum axle load	12.6 tonnes
Tractive effort at start	27 kN per coach
Wheel diameter (new)	890 mm
Wheel base in truck	2450 mm

Half of a four-coach train set ETR 401:

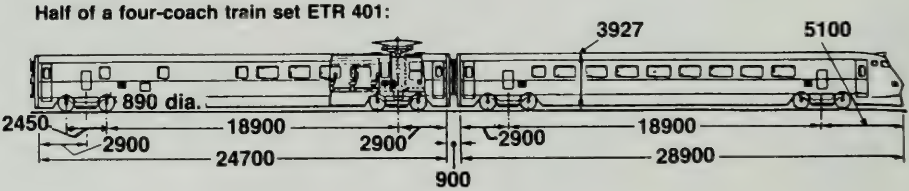

Figure 32

Electric tilting Class ETR 450 (Train configuration 9, Fig. 7)

The ETR 450 is a direct descendant of ETR 401. The concept is the same and many details are identical. However, an ETR 450 train consists of 11 cars of which 10 are powered. The tilt system is the same as that on the ETR 401 with a maximum tilt of 10 degrees. The traction motors are mounted longitudinally below the body between the trucks with transmission to the inner axles of the trucks through cardan shafts.

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Four trains were ordered in May 1985 and a further six in July 1986. The price per train was then $13.4 million. The first ETR 450 is in revenue service since May 29, 1988.

Figure 33 gives some additional information about the ETR 450 train.

Electric train set Class ETR 450 in Italy

Direct current traction motors

10 powered cars + 1 trailer

Maximum speed	250 km/h	Maximum axle load	12.5 tonnes
Power at rail	6250 kW continuously for complete train	Tractive effort at start	230 kN for complete train
		Wheel diameter (new)	890 mm
Power supply	3000 volts DC	Gear ratio	34:63 ≈ 1:1.853
Weight of loaded train	528 tonnes	Wheel base in truck	2450 mm
Weight of empty car	46.5 tonnes		

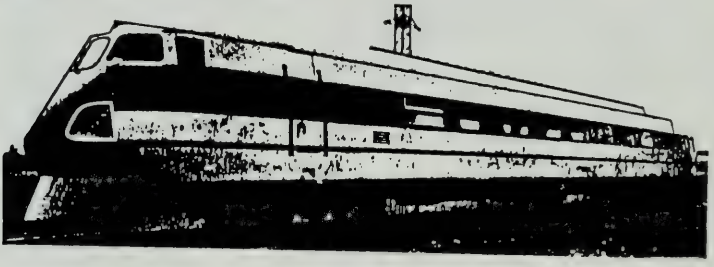

Figure 33

Power car E 404 for train set ETR 500 (Train configuration 3, Fig. 7)

It was announced in 1985 that FS planned to introduce a non-tilting high speed train to be known as ETR 500. It would be designed for a maximum speed of 300 km/h but originally restricted in service to 275 km/h. First a prototype designated ETRX 500 would be designed, constructed and tested and then two pre-series trains named ETRY 500.

The power cars for these trains are designated E 404 and use three-phase traction motors rated 1000 kW continuously and weighing each 2200 kg.

The ETRX 500, consisting of only one power car and one coach, was presented to the media in mid-April 1988. It has been tested up to 260 km/h.

The ETRY 500 trains will have one power car at each end and three intermediate coaches.

The production version, the ETR 500, will basically consist of one power car at each end and between these 11 passenger cars and one service car. On April 28, 1988, FS placed an order with Breda for a pre-series build of two ETR 500 train sets due to enter service in May 1990. Each set will comprise two power cars enclosing 10 coaches. Once the trials are complete, FS intends to order a series of about 100 sets each with two power cars and twelve intermediate cars.

Some details about the E 404 power car are given in Figure 34.

Power car E 404 for electric train set ETR 500 in Italy
Three-phase asynchronous traction motors

Maximum speed	300 km/h (275 km/h service speed)
Power at rail	8000 kW continuously for two power cars
Power supply	3000 volts DC
Weight	624 tonnes for complete train
Maximum axle load	18 tonnes
Tractive effort at start	290 kN for two power cars
Wheel diameter (new)	1100 mm
Wheel base in powered truck	3000 mm

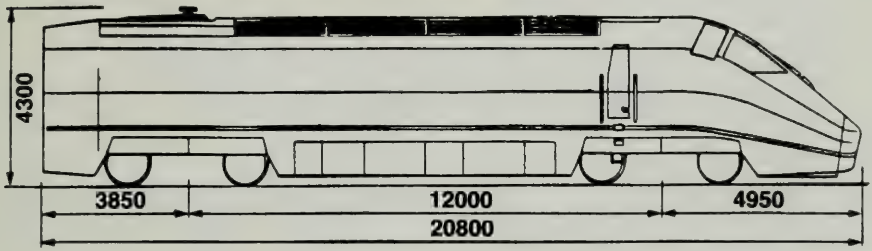

Figure 34

GERMANY

Electric Locomotive Class 103 (Train configuration 1, Fig. 7)

The first four German locomotives geared for a maximum service speed of 200 km/h went into service in 1965. The continuous rating of these prototypes was 5940 kW. As was the general practice in Germany at the time, the traction motors were of the single-phase commutator motor type. In 1970, the first locomotive in a production series of 95 went into service. In comparison with the prototypes, traction motors and transformers had been improved and the continuous output increased to 7080 kW.

Principal data for the production-series Class 103 locomotive are shown in Figure 35. As late as in 1985, a Class 103 locomotive on a test run attained 283 km/h.

Electric multiple-unit train ET 403 (Train configuration 4, Fig. 7)

Each train ET 403 consists of two identical end coaches with passenger compartments, one intermediate passenger coach and one dining car. The train is powered by a total of sixteen compensated DC traction motors, each rated 240 kW and fully spring suspended.

The first train was delivered in 1973 and went into revenue service on April 16, 1974. Only three trains were manufactured, and they are now chartered by Lufthansa for services between the airports in Dusseldorf and Frankfurt.

Figure 36 shows some data for the ET 403 train.

Electric locomotive Class 120 (Train configuration 1, Fig. 7)

After extensive tests with three-phase asynchronous traction motors on three diesel-electric locomotives type DE 2500, the German Federal Railway (DB) decided in 1976 that 5 electric locomotives should be developed with three-phase propulsion and with a continuous rating of 5600

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Electric locomotive Class 103 in Germany
Single-phase commutator-type traction motors

Maximum service speed	200 km/h
Power at rail	7080 kW continuously
Power supply	15 kV, 16 2/3 Hz
Weight	117 tonnes
Axle load	19.5 tonnes
Tractive effort at start	314 kN
Wheel diameter (new)	1250 mm
Gear ratio	1:1.74
Wheel base in truck	2250 + 2250 mm = 4500 mm

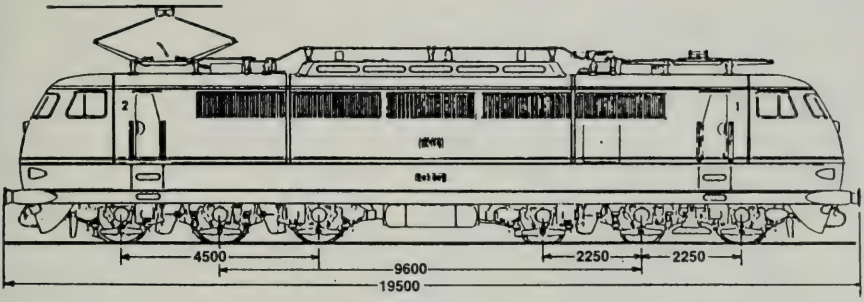

Figure 35

End coach of electric 4-car train set Class ET 403 (Germany)
Direct current traction motors

Maximum speed	200 km/h
Continuous power at rail	3840 kW for the train-set
Power supply	15 kV, 16 2/3 Hz
Weight of train-set in service	236 tonnes
Axle load	15 tonnes
Tractive effort at start	196 kN
Wheel diameter (new)	1050 mm
Wheel base in truck	2600 mm

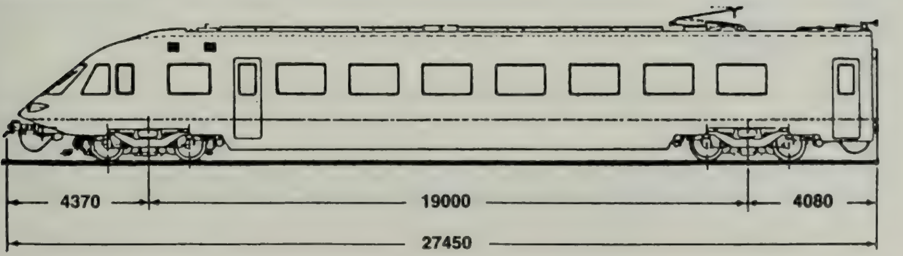

Figure 36

kW. The first locomotive of this Class 120 was formally handed over to DB on May 14, 1979. After it had completed initial trials it was commissioned on February 11, 1980.

One of the five prototype locomotives, number 120 001, was given a gear ratio of 25:103 while the four others used a gear ratio of 22:106. The 120 001 then developed a maximum rotational speed of its traction motors of 3600 r/min for a locomotive speed of 200 km/h with half-worn wheels. It was tested at a speed of 265 km/h on October 17, 1984.

In November 1984, DB ordered 60 locomotives of Class 120 in a production series. The first of these was handed over to DB on January 13, 1987.

Some data for locomotive 120 001 are given in Figure 37.

Electric prototype locomotive 120 001 for West Germany

Three-phase asynchronous traction motors

Maximum speed	280 km/h	Tractive effort at start	290 kN
Power at rail	5600 kW continuously	Wheel diameter (new)	1250 mm
Power supply	15 kV, 16 2/3 Hz	Gear ratio	25:103 1:4.12
Weight of locomotive	84 tonnes	Wheel base in truck	2800 mm
Maximum Axle load	21 tonnes		

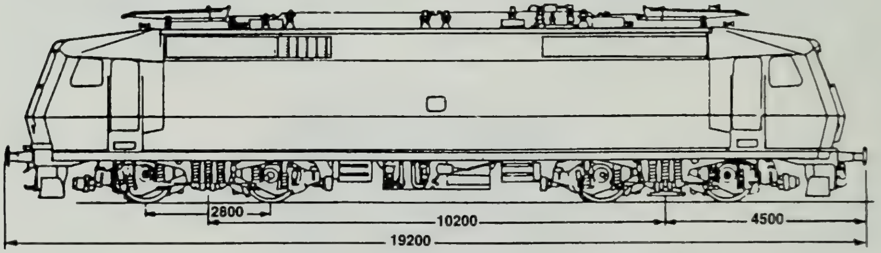

Figure 37

Power cars Class 410 and Class 401 (ICE) (Train configuration 3, Fig. 7)

In parallel with the development of a commercial magnetic levitation train for high speeds, the Transrapid, the DB and the German Federal Ministry for Research and Technology (BMFT) decided in 1982 that also an experimental high-speed train based on the conventional wheel-on-rail concept should be developed. BMFT granted DM 44 million towards the cost of building a four-car research train named Intercity Experimental (ICE) and later designated Class 410. Additional funding from DB and the German industry brought the total funds available to DM 72 million. The actual cost for this project was DM 77 million.

The propulsion for Class 410 was a three-phase drive similar to that used for the Class 120 locomotives. Tests with individual vehicles started in April 1985 and with the whole train in August 1985. Added to the two power cars and two intermediate passenger cars was an equipment test car. Some data for the power cars used in Class 410 are given in Figure 38.

Class 410 attained 317 km/h on November 26, 1985, 345 km/h on November 17, 1986, and 407 km/h on May 1, 1988.

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Electric power car for the German Class 410 high speed train (ICE)
Three-phase asynchronous traction motors

Maximum speed	350 km/h (250 km/h in service)
Power at rail	7280 kW continuously for two power cars
Power supply	15 kV, 16 2/3 Hz
Weight of one power car	78 tonnes
Maximum axle load	19.5 tonnes
Tractive effort at start	270 kN for two power cars
Wheel diameter (new)	1000 mm
Wheel base in truck	3000 mm

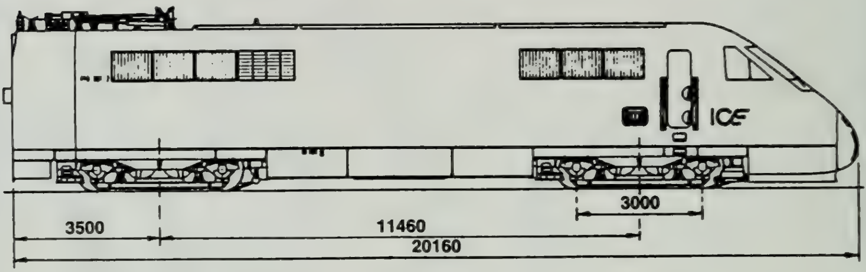

Figure 38

The series-production version, called Intercity Express or officially Class 401 has a higher power rating than Class 410 (Figure 39). The number of intermediate cars between the two power cars is increased to a maximum of 14. DB has ordered 41 such trains and is expected to pay DM 68 million per train. Studies have also been initiated for a freight version of the ICE able to run at 250 km/h.

Power car for the German Class 401 high speed train (IC-Express)
Three-phase asynchronous traction motors

Maximum speed	280 km/h
Power at rail	9600 kW continuously for two power cars
Power supply	15 KV, 16 2/3 Hz
Weight of one power car	78 tonnes
Maximum axle load	19.5 tonnes
Tractive effort at start	400 kN for two power cars
Wheel diameter (new)	1000 mm
Wheel base in truck	3000 mm

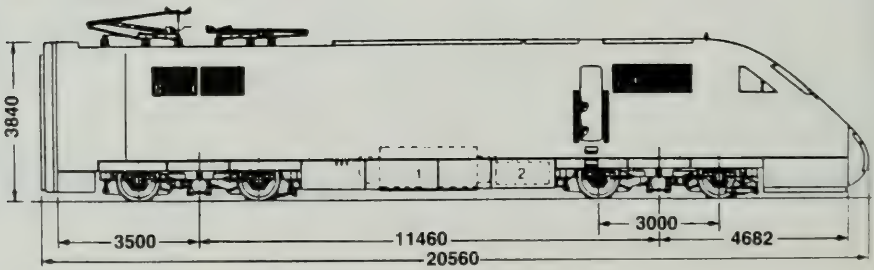

Figure 39

BRITAIN

Diesel-electric power car Class 253 (Train configuration 3, Fig. 7)

Class 253 is also known as HST (High Speed Train). The first production version went into service on October 4, 1976. Normally, a Class 253 train comprises one diesel-electric power car at each end and seven intermediate passenger cars. The 12-cylinder diesel engine in each power car has 1453 kW available for traction at a relational speed of 1500 r/min. It drives a three-phase main alternator rated 1480 kW. The frame-mounted DC traction motors are each rated 343 kW and are permanently connected in series-parallel.

A total of 95 trains of this class have been delivered to British Rail. Figure 40 gives some details of the Class 253 power car. One of these trains has attained 234 km/h on a test run.

Diesel-electric power car Class 253 (HST) for Britain

Direct current traction motors

Maximum speed	200 km/h	Axle load	16.5 tonnes
Diesel engine rating (UIC)	1678 kW at 1500 r/min	Tractive effort at start	89 kN per power car
Diesel power for traction	1455 kW	Wheel diameter (new)	1020 mm
Weight of power car	66 tonnes	Wheel base in truck	2600 mm

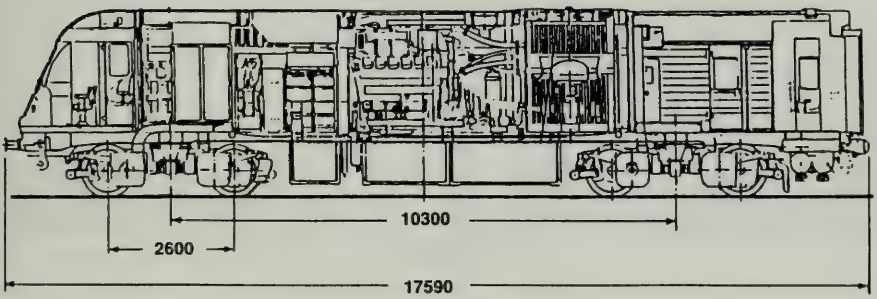

Figure 40

Electric locomotive Class 89 (Train configuration 1, Fig. 7)

The Class 89 locomotive is geared for a maximum speed of 200 km/h. It was originally intended as a standard passenger locomotive for the East Coast main line in Britain, but later developments and especially the request for even higher speeds on main routes in Britain and on the European continent have made it unlikely that the only prototype Class 89 built, ordered in 1983 and delivered in 1986, will be followed by a series production.

The Class 86 locomotives in common use in Britain are geared for a maximum speed of 160 km/h and have an axle load of 21 tonnes. This has been blamed for track damage on high speed sections of the West Coast main line, and therefore the axle load of Class 89 was specified to only 17.5 tonnes. Figure 41 shows some data for the Class 89 locomotive.

Electric locomotive Class 91 (Train configuration 1, Fig. 7)

Preliminary designated as IC 225 or Electra, this type of locomotive has finally been named Class 91. Formal contracts for 31 locomotives purchased from the British GEC were signed on October 1,

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Electric locomotive Class 89 for Britain
Direct current traction motors

Maximum speed	200 km/h	Tractive effort at start	
Power at rail	4350 kW continuously	Wheel base in truck	2100 + 2300 mm
Power supply	25 kV, 50 Hz		= 4400 mm
Weight of locomotive	105 tonnes	Wheel diameter (new)	1070 mm
Axle load	17.5 tonnes		

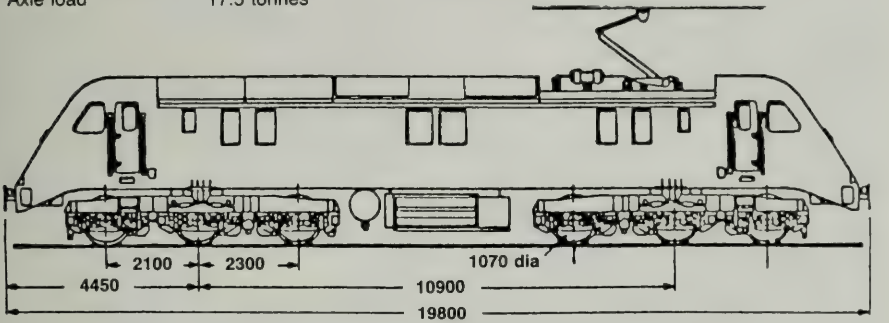

Figure 41

1986. Class 91 is apparently intended as British Rail's standard high speed electric locomotive but it still uses thyristor-controlled separately excited DC traction motors.

The two traction motors in each track are suspended from the locomotive body. This minimizes the weight of the truck and limits the stresses imposed on the track. The motors drive truck-mounted gearboxes by means of cardan transmission shafts.

The axle load is 20.5 tonnes and the wheel base in truck no less than 3350 mm.

The first locomotive was delivered from the manufacturer on February 12, 1988.

Some data for the Class 91 locomotive are shown in Figure 42.

Electric locomotive Class 91 (Electra) for Britain
Direct current traction motors, separately excited

Maximum speed	225 km/h	Weight of locomotive	82 tonnes
Continuous power at rail	4530 kW at 153 km/h	Axle load	20.5 tonnes
	3750 kW at 225 km/h	Tractive effort at start	193 kN
Power Supply	25 kV, 50 Hz		
Weight of locomotive	82 tonnes		

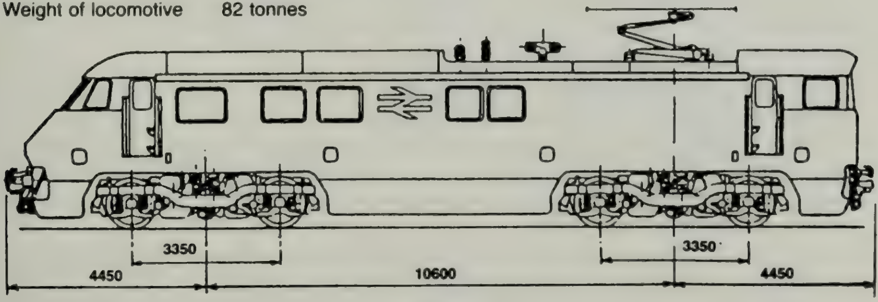

Figure 42

THE SOVIET UNION (USSR)

Electric locomotive consist Class ChS 200 (Train configuration 2, Fig. 7)

After the end of World War II, the USSR decided to build domestically electric locomotives for freight transportation only while electric locomotives for passenger services have been imported mainly from the Czechoslovakian manufacturer Skoda. It was decided in 1973 that for a maximum speed of 200 km/h a locomotive consist with a total of eight axles, arrangement BoBo + BoBo, should be developed by Skoda and designated ChS 200. The first two consists were delivered in 1975 and since then at least 20 more have been delivered but half of those geared for only 160 km/h.

The ChS 200 locomotive (Skoda's type 66E) are powered by fully suspended DC series-wound traction motors from Skoda. They are mainly intended to haul express passenger trains of 650 tonnes on the Moscow-Leningrad line. By comparison with the ER 200 multiple-unit trains the Soviet Railways hope to be able to decide whether locomotive-hauled or multiple-unit operation is the best option for this service.

Skoda is believed to have delivered at least 300 Class ChS 200 locomotives to the USSR for various services. The axle load is 19 tonnes and the wheel base in truck 3200 mm, and some additional parameters are shown in Figure 43.

Electric locomotive consist Class ChS 200 for the USSR

Direct current traction motors

Maximum speed	200 km/h	Axle load	19 tonnes
Power at rail	8000 kW continuously	Tractive effort at start	420 kN
Power supply	3000 volts DC	Wheel diameter (new)	1250 mm
Weight of locomotive consist	152 tonnes	Wheel base in truck	3200 mm

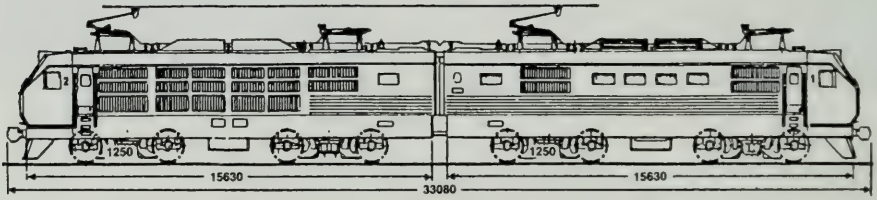

Figure 43

Electric multiple-unit train Class ER 200 (Train configuration 8, Fig. 7)

The trunk line between Moscow and Leningrad is the most important for intercity passenger traffic in the USSR. Accordingly, it was the first railroad in the Soviet Union to be provided with train sets able to run at speeds up to 200 km/h. The multiple unit train sets Class ER 200 have been designed and built by the Rigs Carriage Works in Latvia.

The ER 200 is a 14-unit train with a maximum speed of 200 km/h. It has a rather unusual make-up with the end cars having driver's cabs and buffet areas but no powered axles while the intermediate twelve cars are permanently formed into six identical two-car groups in which all axles are powered. For the whole train, the continuous power at rail is no less than 10320 kW, and the total weight is 830 tonnes of which 720 tonnes can be used for adhesion. Maximum axle load is 15 tonnes for empty train and 17 tonnes for the train fully loaded.

The conventional DC traction motors are fully suspended and each rated 215 kW continuously. The wheel base in truck is 2500 mm for both powered and non-powered trucks.

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The ER 200 entered service the first time between Moscow and Leningrad on November 6, 1977, in existing time schedule, but not until March 1, 1984, could the planned increase in speed be achieved. Average speed is 140 km/h between Moscow and Leningrad.

Figure 44 shows some details related to train ER 200.

Trailer with driver's cab for train Class ER 200 in the USSR

Train consists of 1 trailer + 12 powered cars + 1 trailer
Direct current traction motors

Maximum speed	200 km/h
Power at rail	48 × 215 kW = 10320 kW for complete train
Power supply	3000 volts DC
Weight of powered car	60 tonnes
Maximum axle load	17 tonnes fully loaded
Wheel diameter (new)	950 mm
Gear ratio	1:2.35
Wheel base in powered truck	2500 mm

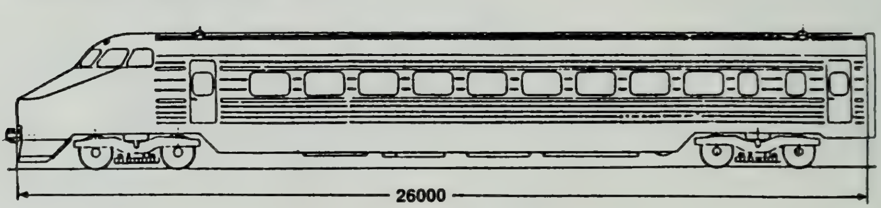

Figure 44

THE UNITED STATES (USA)

Electric locomotive Class AEM 7 (Train configuration 1, Fig. 7)

In the mid-1970's Amtrak decided to lease two European-built electric locomotives to be tried out in the Northeast corridor. The choice fell on a modified Rc 4 locomotive from ASEA of Sweden and a modified CC 21000 locomotive from Alsthom and Francorail in France. After three months of testing in 1977, Amtrak concluded that a fair test of the French locomotives capabilities could not be performed under the poor track conditions that then existed on much of the Northeast Corridor. The Swedish locomotive completed successfully an extended testing period.

Early in 1978, Amtrak placed an order for the first eight new locomotives now designated AEM 7. Due to Amtrak's request that the locomotive should be able to run on three different catenary systems, a number of modifications were necessary relative to the prototype locomotive, and the principal parameters of the AEM 7 are shown in Figure 45.

Separately excited compensated DC traction motors are used for the AEM 7.

Multi-system electric locomotive Class AEM 7 for USA
Direct current traction motors

Maximum speed	200 km/h	Weight of locomotive	91 tonnes
Power at rail	4320 kW continuously	Axle load	22.8 tonnes
Power supply	11 kV, 25 Hz	Tractive effort at start	230 kN
	12.5 kV, 60 Hz	Wheel diameter (new)	1300 mm
	25 kV, 60 Hz	Wheel base in truck	2760 mm

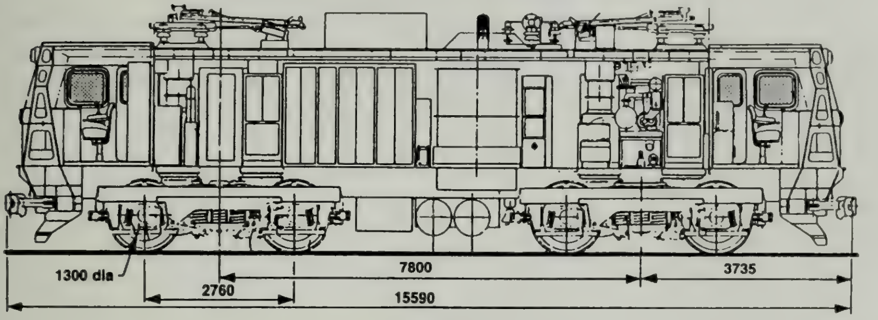

Figure 45

Diesel-electric locomotive Class F 69 PH-AC (Train configuration 1, Fig. 7)

In order to get experience with a high-speed diesel-electric locomotive using three-phase propulsion, Amtrak has ordered two such locomotives Class F 69 PH-AC from General Motors Locomotive Group. Development and supply of electrical equipment has been the responsibility of Siemens in Germany.

Some principal parameters of this type of locomotive are shown in Figure 46.

The first locomotive was presented to the media at a roll-out procedure on July 12, 1989.

Diesel-electric locomotive model F69PH-AC for Amtrak
Three-phase asynchronous traction motors

Maximum speed	177 km/h	Diesel engine rating (AAR)	2260 kW at 900 r/min
Axle arrangement	Bo'Bo'	Diesel engine type	GM 12-710 G3
Weight	120 tonnes	Electrical control system	Voltage source inverter with GTO thyristors
Axle load	30 tonnes		
Tractive effort at start	276 kN		

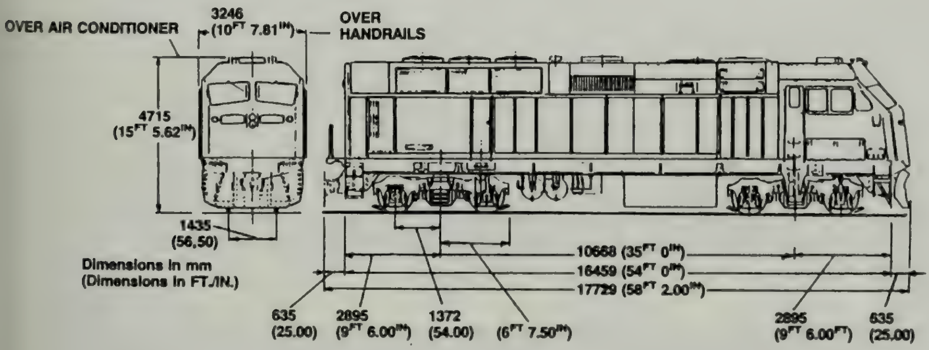

Figure 46

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CANADA

Diesel-electric power car model LRC (Train configuration 3 or 1, Fig. 7)

The LRC, where the acronym stands for Light, Rapid, Comfortable, is a trainset normally consisting of one power car at each end with ten intermediate coaches, or sometimes one power car (a locomotive) hauling five coaches. The train is designed for a maximum speed of 193 km/h.

One power car and one coach were tested at Pueblo at the end of 1974 and then entered service between Toronto and Sarnia on March 3, 1975. A Canadian train speed record was set on March 10, 1975, when the LRC reached 205 km/h.

The LRC power car has a low center of gravity and a comparatively low profile, and the front end is designed for low air resistance. The underframe is designed with a depressed box to accommodate the diesel engine. There is no provision for tilting the body of the power car, but the coaches can be hydraulically tilted up to a maximum of 10 degrees. The power car has a cross-section of only 9.57 m^2. Further details are shown in Figure 47.

Diesel-electric power car model LRC for Canada

Direct current traction motors

Maximum speed	193 km/h	Axle load	24.7 tonnes
Diesel engine rating	2163 kW at 900 r/min	Tractive effort at start	216 kN per power car
Diesel power for traction	1715 kW	Wheel diameter (new)	1016 mm
Weight of power car	98.8 tonnes	Wheel base in truck	2896 mm

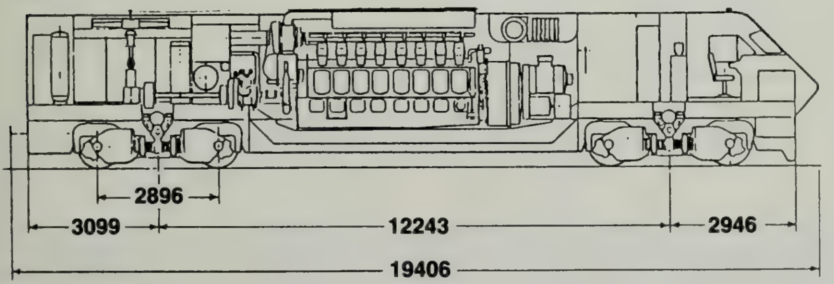

Figure 47

SPAIN

Diesel-hydraulic locomotive Class 354 (Train configuration 1, Fig. 7)

Since 1964, trains of the TALGO type have been hauled on non-electrified lines by diesel-hydraulic locomotives built by German manufacturers. The latest version of these is Class 354 of which eight units have been delivered from Krauss-Maffei. Each unit contains two diesel engines with a UIC rating of 1535 kW each, at 1900 r/min. Originally these locomotives were geared for a maximum speed of 180 km/h, and the first was commissioned at the end of 1982. Later the maximum speed was increased to 200 km/h and a decision was made in 1988 to modify at least one locomotive for 250 km/h test running.

Figure 48 shows some details of the Class 354 locomotive.

Diesel-hydraulic locomotive Class 354 for Spain

Used for hauling TALGO trains mainly on the RENFE network

Maximum speed	200 km/h	Tractive effort at start	260 kN
Track gauge	1668 mm	Total continuous diesel engine rating (UIC)	3070 kW at 1900 r/min
Axle arrangement	B'B'	Two diesel engines of type	MTU 16V 396 TD 13
Weight	80 tonnes	Two hydraulic torque converters of type	Voith L 520 rz U2
Axle load	20 tonnes		

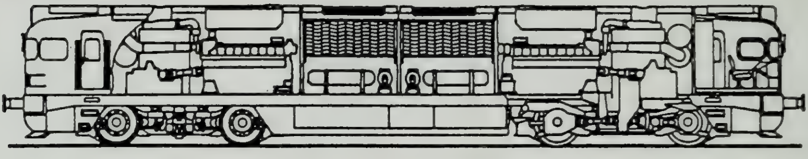

Figure 48

SWEDEN

Electric high speed train X 2 (Train configuration 1, Fig. 7)

Already in 1973 the Swedish State Railways (SJ) and ASEA (now ABB) signed an agreement to develop a train that could, with the help of active body-tilting, run at speeds exceeding 200 km/h, even through existing curves with radii down to 1000 m. Tests on older modified trains encouraged SJ to request international bids for such high-speed trains to meet very tough specifications, but it was only after SJ relaxed the requirement on an axle load of 15 tonnes to instead read 17.5 tonnes that it was possible for a bidder to meet the other parts of the specification and only by applying three-phase traction instead of DC traction motors.

In August 1986, ABB received an order for 20 high speed trainsets X 2 with an option for 32 additional sets. Train X 2 embodies three technical characteristics of importance for its use on existing curved track:

(a) Trucks with radial steering axles
(b) An active tilting system for the coaches
(c) A propulsion system with three-phase asynchronous traction motors

The X 2 train will in principle consist of one power car and five coaches. The coach at the end of the train will be provided with a driver's cab from which the train can also be monitored.

Figure 49 shows some of the principal parameters of the Class X 2 train.

SWITZERLAND

Electric locomotive Class Re 4/4 VI (Train configuration 1, Fig. 7)

The Swiss Federal Railways (SBB) placed in 1988 an order with ABB (electrical equipment) and SLM (mechanical part) for 12 general-duty express locomotives. With three-phase propulsion developed by ABB and new locomotive body and truck construction designed by SLM, the Re 4/4 VI represents a new generation of traction vehicles. With a continuous rating of 6100 kW it will become the most powerful 4-axle locomotive in the world.

Some details of the locomotive are shown in Figure 50. The first unit is scheduled for delivery in 1990.

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Electric power car for high speed train Class X 2 (Sweden)
Three-phase asynchronous traction motors

Maximum serivce speed	200 km/h	Maximum axle load	17.5 tonnes
Continuous power at rail	3260 kW	Tractive effort at start	160 kN
Power supply	15 kV, 16 2/3 Hz	Wheel diameter (new)	1100 mm
Weight of power car	70 tonnes		

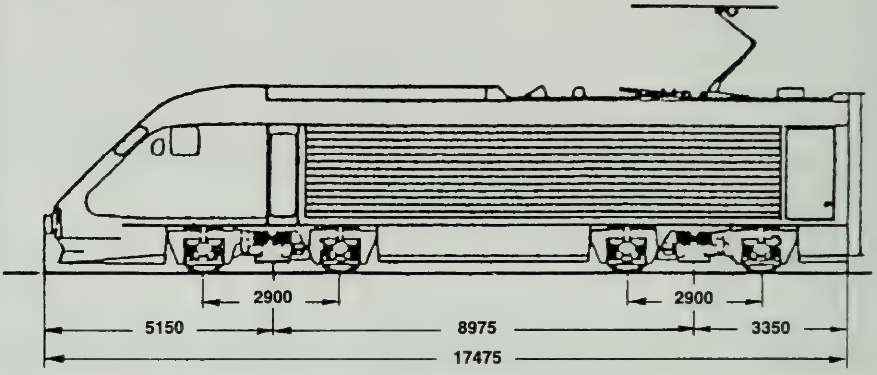


Figure 49

Electric locomotive Class Re 4/4 VI for Switzerland
Three-phase asynchronous traction motors

Maximum speed	230 km/h	Axle load	20 tonnes
Power at rail	6100 kW continuously	Tractive effort at start	
Power supply	15 kV, 16 2/3 Hz	Wheel diameter (new)	1100 mm
Weight of locomotive	80 tonnes	Wheel base in truck	2800 mm

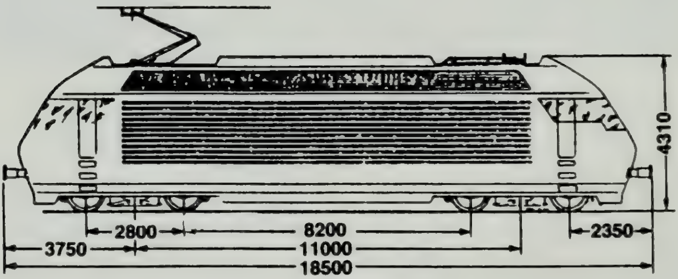


Figure 50

COMMENTS AND CONCLUSIONS

A survey implies a systematic collection and analysis of data for a specific purpose. The preceding part of this paper represents the collection of information relative to practically all wheel-on-rail traction vehicles in the world capable of running at high speed as defined at the beginning of the survey.

An analysis of the information compiled yields a number of interesting results, some of which are listed below.

Train resistance

Before traction vehicles for very high speeds became available the actual train resistance at these speeds could not be accurately measured, only estimated or extrapolated from measurements at lower speeds. Actual investigations on the Class 23000 trainsets are shown in Figure 51 together with a subdivision of the total resistance into its three parts in the Davis' formula and also the impact of various components of the train on the resistance.

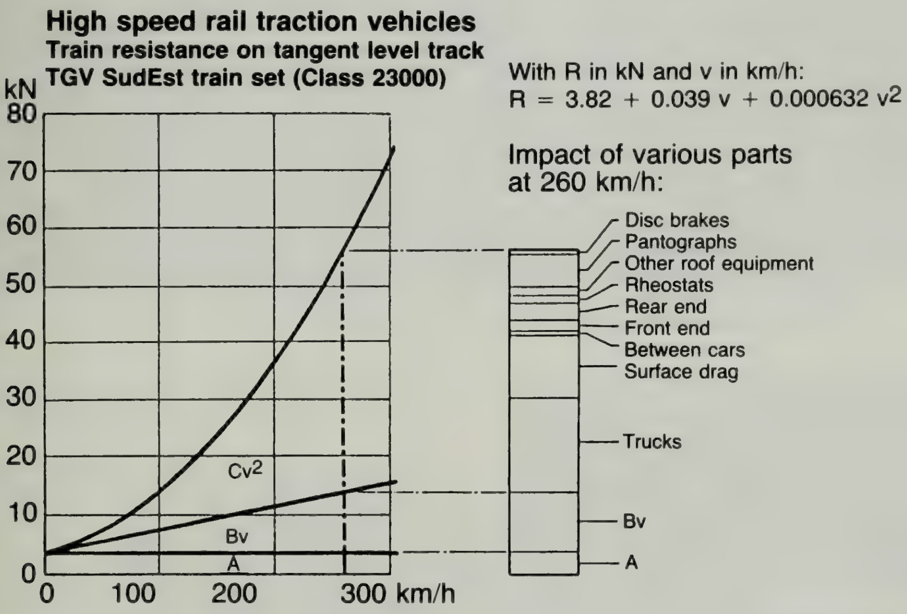


Figure 51

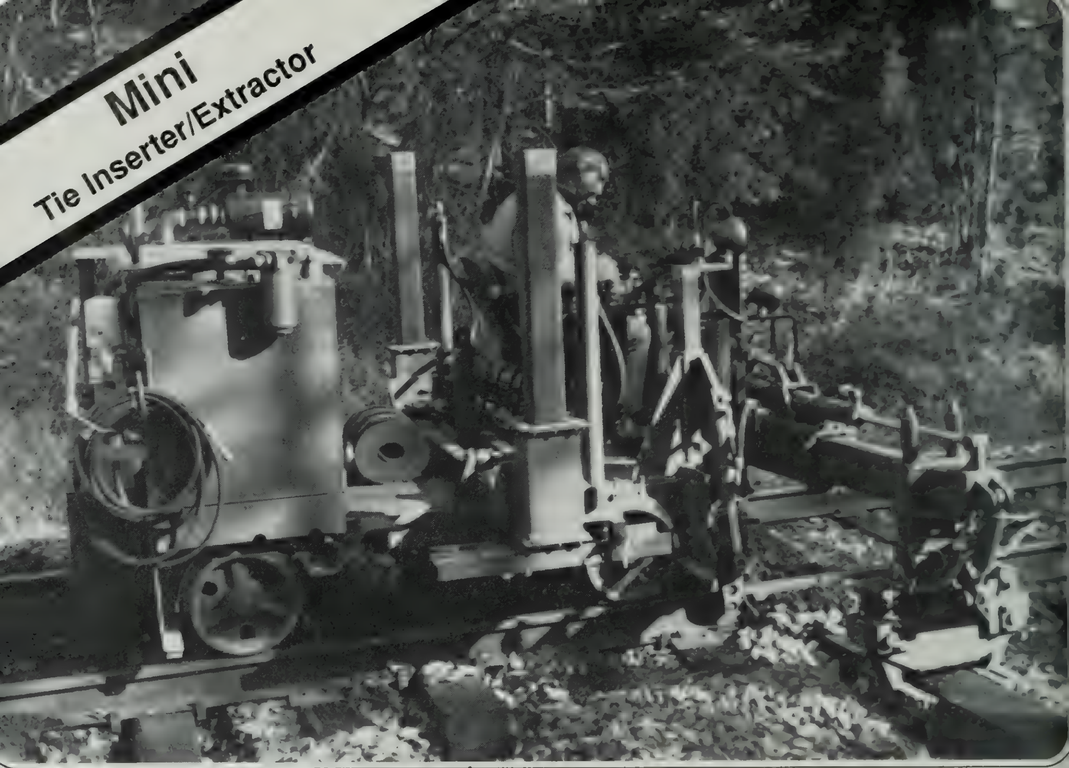
Power requirement

From the measurements quoted above, the power needed at the wheel-rail interface for balanced speed on tangent level track can be calculated for various speeds up to 300km/h (Figure 52). The curve shown corresponds, of course, to a train shaped as the Class 23000, but in most cases the necessary power might be even higher for other trains since Class 23000 has an almost ideal shape and a rather small cross-section. With a power needed *at rail* of 6000 kW at 300 km/h, it is evident that any diesel-powered traction vehicle will have great difficulty in achieving a maximum speed of much more than 200 km/h.

Wheel base in truck

Figure 53 shows the actual wheel base in two-axle trucks for the traction vehicles described earlier. The wheel base is plotted against time (year of delivery for the vehicle). Most electric multiple-unit cars still have a wheel base in truck of about 2500 mm, while for locomotives and power cars there is a trend to increase the wheel base to a range of 2800 to 3000 mm.

Mini
Tie Inserter/Extractor



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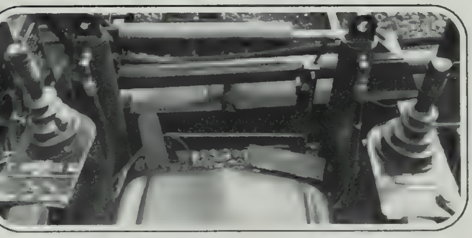
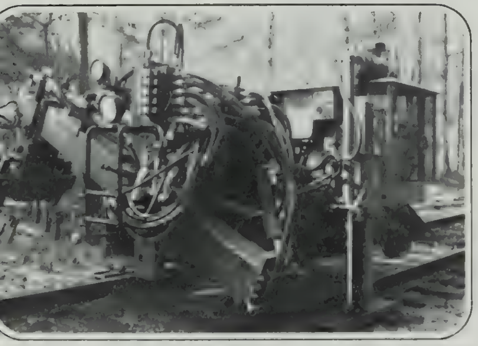


ully proportional electronic joysticks or optimum control during tie perations.



Unit in raised position ready to "walk" ff track.

- Joystick controls give precise proportional control over tie extracting/inserting operations.
- Boom lifts 25 degrees above horizontal — 15 degrees below horizontal. Slew movement is 15 degrees to accomodate slewed or switch ties and for tie change-out.
- Optional "walking beam" set off clears the right-of-way in six minutes, and will operate on inclines up to 30 degrees.
- Removes/inserts up to 55 ties per hour.
-

Frame:	Welded tubular steel	
Length:	124"	(3,150 mm)
Width:	107"	(2,718 mm)
Height:	83"	(2,108 mm)
Weight:	7000 pounds	(3,175 kg)
Wheelbase:	60"	(1,524 mm)

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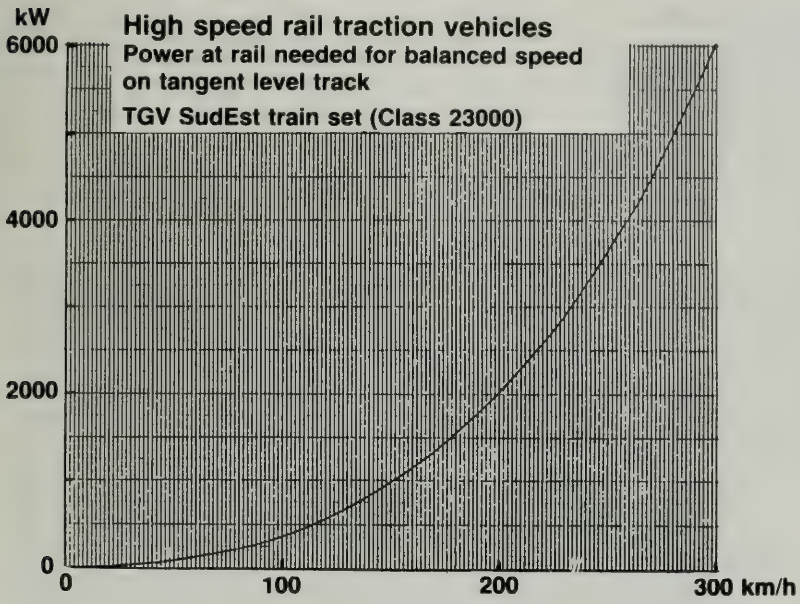

Figure 52

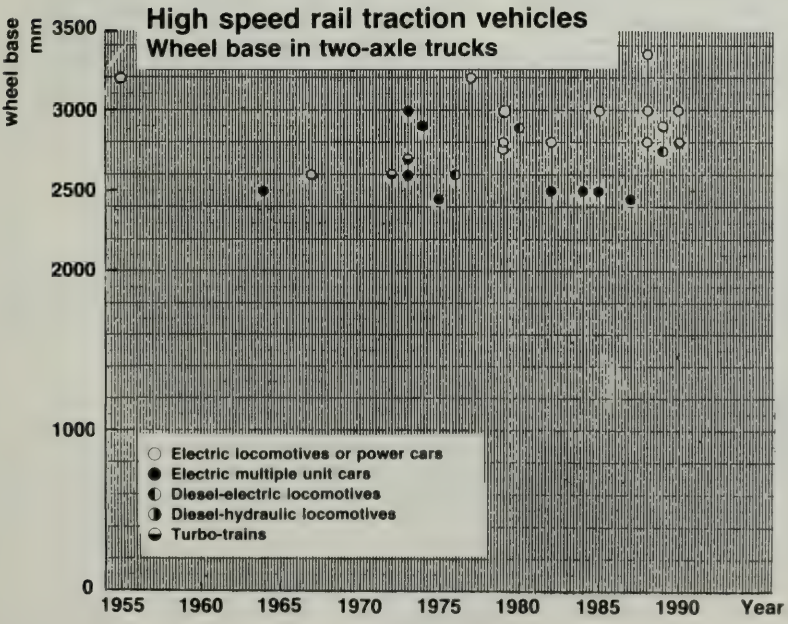

Figure 53

Maximum axle load

In Figure 54 the maximum axle load of the traction vehicles treated earlier has been plotted against year of delivery. The state of the art is approximately that the following maximum axle loads are achievable for high speed traction vehicles of different categories:

16 to 17 tonnes for electric multiple-unit cars
17 to 20 tonnes for electric power cars
20 to 21 tonnes for electric universal-type locomotives

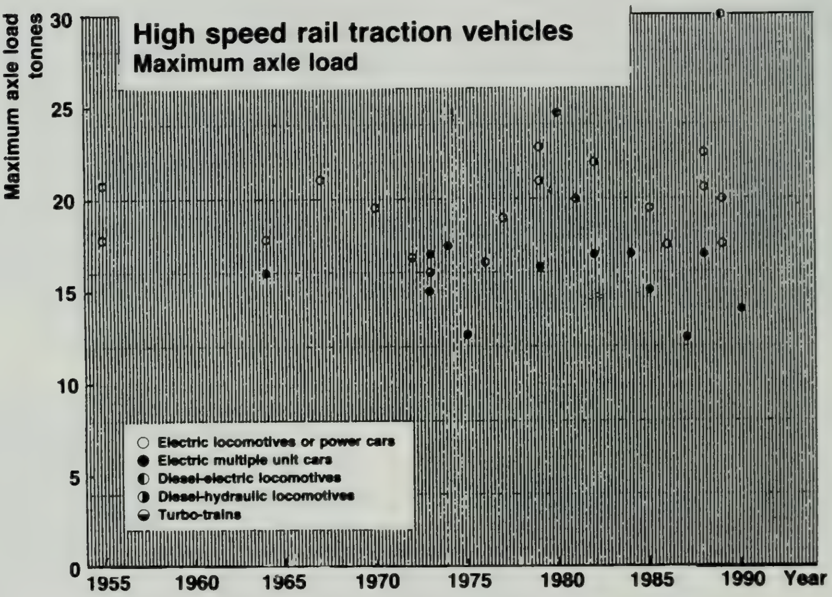

Figure 54

Power to weight ratio of traction vehicles

In Figure 55 is plotted the continuous power at rail divided by the weight of the traction vehicle against the year of delivery for the vehicle. The superiority of electric locomotives and power cars over all other types of traction vehicles is striking, and the increase over the last several years is mainly due to the introduction of three-phase propulsion.

Propulsion systems

The use of three-phase propulsion mentioned in the last paragraph is not and should not be restrained to electric locomotives and power cars. The concept is applicable to many types of traction vehicles as shown in Figure 56. Three-phase propulsion is likely to replace most of DC traction motors over the next decade.

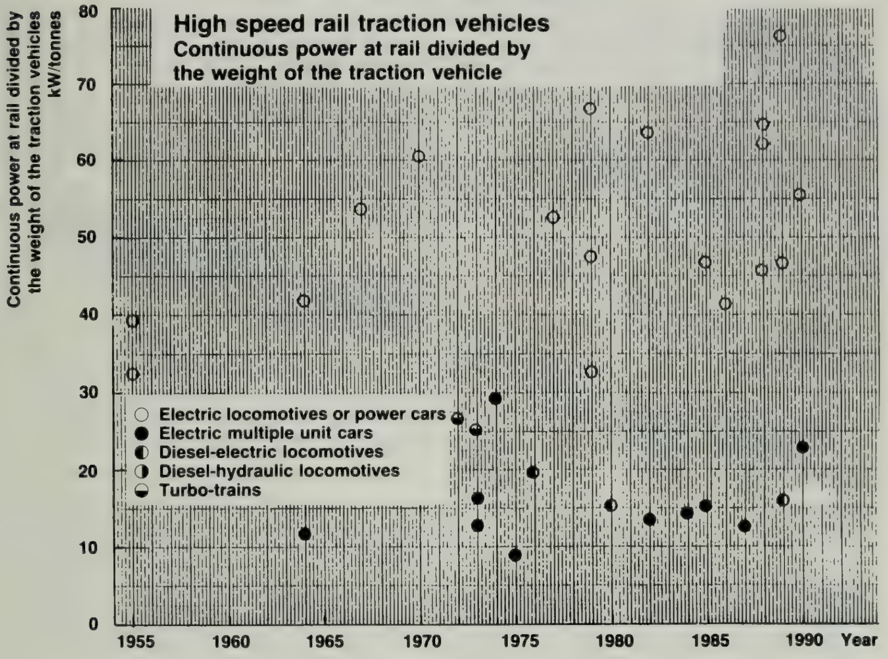

Figure 55

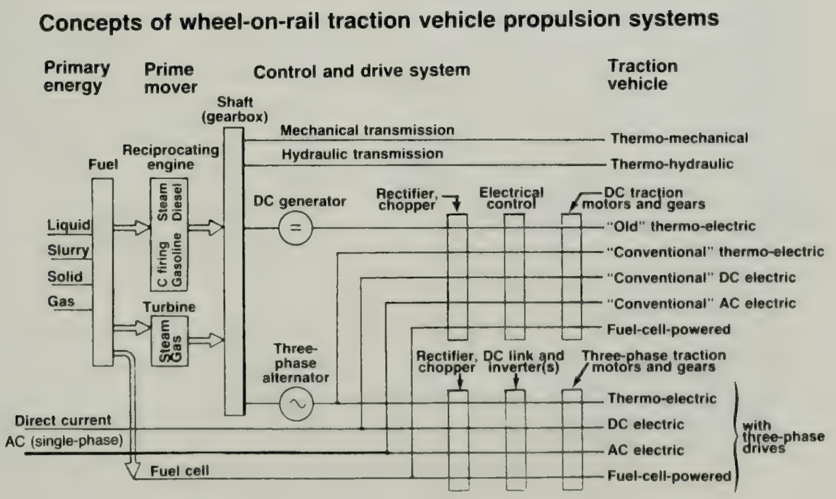

Figure 56

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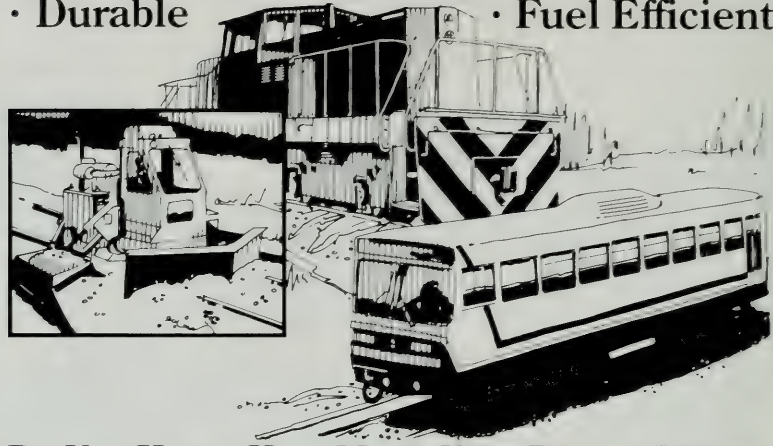

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Prices of high speed trains

In Figure 57 are listed approximate prices for some of the high-speed trainsets described earlier. These prices should be treated with some caution. The rate of exchange between the currency in the contract and the US dollar may vary considerably over time, and the prices are, of course, dependent on quantity of trains manufactured on the same order.

Approximate prices of high speed trains

Train	Price per train US$ millions	Based on number of trains	Year
ETR 450 (Italy)	22	6	1986
ICE (Germany)	24	41	1988
TAV (Spain)	18	24	1989
TGV Atlantique	21	95	1989
TGV Paris-SudEst	13	109	1988
X 2 (Sweden)	11	20	1986

Figure 57

Infrastructure parameters for high speed rail

Although not directly a part of a survey of traction vehicles, some infrastructure parameters have been listed in Figure 58 to indicate what has been achieved or is being planned for important high speed lines.

Some infrastructure parameters for high speed rail

		Japan		Italy	France	West Germany		USSR
		Shinkansen		Direttissima	TGV	Hannover –	Köln –	Moscow
		Tokaido	Sanyo....			– Würzburg	– Frankfurt	South
Design speed	km/h	210	260	250	300	250	250	250
Weight of rail	kg/m	53	60	60	60	60	60	60
Minimum curve radii								
Horizontally	m	2500	4000	3000	4000	7000	3250	4000
Vertically	m	10000	15000	30000	25000	25000	25000	
Maximum superelevation	mm	180	110 – 155	125	180		230	
Ruling gradient	%	2.0	1.5	0.8	3.5 1.5	1.25	2.5	1.5
Distance between track centers	mm	4200	4300	4000	4200	4700		
Width of double track subgrade	m	10.7	11.4–11.6	11.0	13.0	15.7		
Work started	Year	1959	1968	1970	1978			
In operation	Year	1964	1972	1977	1981			

Figure 58

Overhead contact systems for high speed rail

Figure 59 shows the main parameters for overhead contact systems in use for important high speed lines.

Some overhead contact systems for high speed rail

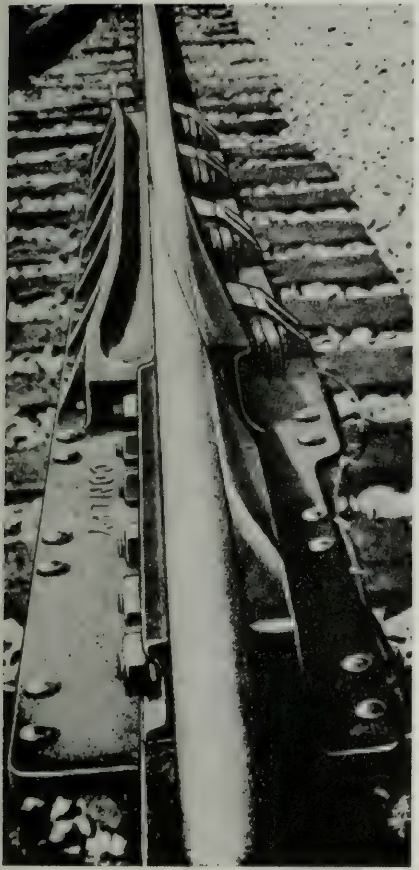
		Japan		France	West Germany
		Tokaido	Sanyo	TGV	
Voltage	kV	25	25	25	15
Frequency	Hz	60	50 or 60	50	16 2/3
Catenary type		Composite compound	Heavy compound	Stitched simple	Stitched simple
Standard span	m	50	50	63	65
Contact wire					
Material		Copper	Copper	Copper	Copper
Section	mm^2	110	170	120	120
Tension	kN	10	15	14	15
Messenger wire					
Material		CdCu	Steel	Bronze	
Section	mm^2	80	180	65	70
Tension	kN	10	25	14	19
Auxiliary messenger wire				None	None
Material		CdCu	Copper		
Section	mm^2	60	150		
Tension	kN	10	15		
Stitch		None	None		
Material				Bronze	Bronze
Section	mm^2			35	35
Tension	kN			4	3.5
Length	m			15	18
Droppers					
Material				Copper	
Section	mm^2			d = 5 mm	16
Spacing	m	5.0	5.0 (3.5)	4.5	9.0
Height of catenary assembly	m			1.4	1.8
Height of contact wire over rail	m			4.90	5.30

Figure 59

The planned European high speed rail network

Network		V 1	V 2	V 3
Year		1995	2005	2015
New lines	km	5200	7100	9100
Upgraded lines	km	7100	8400	9900
Link lines	km	—	—	6500
Feeder lines	km	—	—	4500
Grand total	km	12300	15500	30000
Investments in billions ECUs				
Infrastructure		43	58	90
Rolling stock		10	20	30

Figure 60



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THE PLANNED EUROPEAN HIGH SPEED RAIL NETWORK

It may be of general interest in this context to conclude this paper with a few words about the extensive high speed rail network that will emerge in Europe gradually over the period until the year 2015. Three phases have been defined, named V1, V2 and V3 and ending in theyears 1995, 2005 and 2015, respectively. The planned route-kilometers and investments are indicated in Figure 60. The estimated economic rate of return on the investment is shown in Figure 61 and a map of the high speed rail network in Figure 62.

The emergence of this European network at a total cost of about US $120 billion should be cause for a new look at transportation policy also in other parts of the world.

European high-speed ground transportation network

Estimated economic rate of return

Network	Year of commissioning	Ten years after commissioning
V 1	9.4% in 1995	13.3% in 2005
V 2	9.8% in 2005	14.1% in 2015
V 3	10.8% in 2015	15.5% in 2025

Figure 61

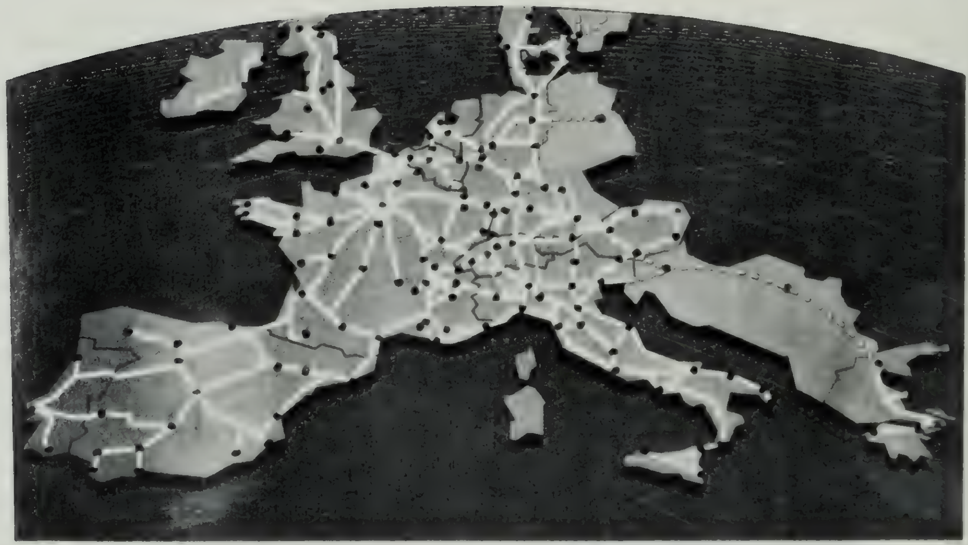

Figure 62

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STANDARDIZATION OF TURNOUT MATERIAL

Subcommittee Chairman: H. G. Webb

The idea of standardization of track material among the United States railroads has been discussed, studied and "discarded" many times over many years. Each railroad has its own standards developed through the years of past practices. Each is positive it has the best possible design. Each continues to stand forthrightly against any change towards an industry standardization of almost anything. There must be a compelling economic effect for each railroad to effect any change in a railroad's standards.

The only standardization that has progressed much is the rail section. Even in this small movement towards standardization in rail usage, there still remains three predominant mainline rail weight sections with little movement in an effort to standardize.

Turnout material standardization is one area of railroad maintenance expense that has also been studied over the years. It is an area of each railroads basic cost that continues to grow due to the increased wheel loads and increased volume of traffic. It is an area of standardization that should be progressed and perhaps the first of many other standardizations. There appear to be great opportunities for savings to the railroads and to the manufacturers of turnout material if standardization could be achieved.

Once again, each railroad has its own designs, weight of rail, specifications for metallurgy, frog configurations, switch point designs, stock rail design and metallurgy, and so on. There will have to be an impelling reason for railroads to get together to standardize turnout material. The economy that may be achieved could be the catalyst to begin this standardization. To gain the economies, it would appear all major railroads would have to agree to progressing the standardization developed.

The following investigation is made to determine the economics that could occur in the "Standardization of Turnout Material."

The approach used in this study is as follows:

1. Calculate the total number of turnouts in eight leading U.S. railroads.
2. Calculate the estimated cost of renewal of each turnout in service on those railroads.
3. Estimate the cost of annual maintenance of turnouts.
4. Estimate the replacement cost of complete turnouts.
5. Through a questionnaire to manufacturers and railroaders, derive the estimated savings that could be achieved through standardization.
6. Through evaluating a yearly cost of turnout maintenance and replacement costs, calculate the net present value of standardization.

These calculations do show there is some savings to the industry and to each individual railroad in the standardization of turnout material. The study shows an approximate $10.4 million total annual savings with a net present value over the cycle life of 20 years of $87.2 million. Each railroad's total portion increases with their size and use of replacement materials both for yearly maintenance and life cycle renewal. Thus, there is adequate support, even though monetary savings were not as great as anticipated, for railroads to begin discussions of the details of that standardization. These discussions, to be fruitful, must be carried out at levels above the railroad's design engineers. (Each railroad's engineers tend to have considerable ownership to their railroad's specifications; thus an ingrown resistance to change.) These discussions must be done with an open mind and with a receptive attitude towards change for long range economic benefits.

In calculating the industry's (eight railroads shown in Table 1) total number of turnouts, several assumptions were made. Since it was nearly impossible to collect the exact number and size of turnouts each railroad has in service, certain assumptions were made and a calculation was made and is shown in Table 1. The exact details of one railroad's number of turnouts and sizes was known so a direct proportion per total mile of track maintained was used. The miles maintained by each road came from the 1987 R-1 Report. Since the precision of these numbers is not acutely necessary, it was thought these assumptions would give a good approximation of the number of turnouts we are dealing with.

Also shown in Table 1 is a calculation of the total replacement value of the turnouts totaled on the selected eight U.S. railroads. Once again, an assumption was made to use one railroad's known cost of material in replacing a turnout for each size in the study. Then it was assumed about 10% of that cost was considered track material that would not be affected by standardization.

Table 1
Railroads T.O. Replacement Value

Railroad	Mls. Mtn.	No. 10/12	No. 15/16	No. 20/24	Other	Est. Total T.O.'s
BN	23,476	13,130	1,873	1,779	3,123	19,900
UP	20,944	11,715	1,668	1,580	2,787	17,800
CSX	18,030	10,084	1,437	1,360	2,399	15,300
CR	11,215	6,273	893	846	1,492	9,500
NS	15,425	8,627	1,230	1,163	2,052	13,100
SF	11,709	6,544	934	874	1,559	9,900
SP	9,901	5,538	789	747	1,317	8,400
CNW	5,486	3,068	437	414	730	4,600
TOTAL	**116,186**	**64,979**	**9,250**	**8,754**	**15,459**	**98,500**

Estimated Cost T.O. Material

	Total	Steel Only (90%)
No. 10/12	$14,000	$12,600
No. 15/16	$18,000	$16,200
No. 20/24	$23,000	$20,000

Total replacement value effected by standardization of the steel portions (non OTM) of T.O.

	(In Millions)
No. 10/12	$ 818.7
No. 15/16	$ 149.9
No. 20/24	$ 181.2
Total	**$1,149.8**

Mr. Bill Hamilton, Consultant, under contract to the AAR, developed a thorough study of several railroad's annual material replacement cost used in maintaining turnouts. His study derived a $506 per year per turnout material cost. His approach and results were published in the AAR report dated October 31, 1986. They were also published in a Railway Track and Structures article in their issue of February, 1988. The study points out that most railroads have never calcualted their turnout maintenance cost. Using the derived $506 per turnout, it can be seen that the eight railroads surveyed are spending $49.8 million per year in maintaining these turnouts.

To determine the possible savings involved in the standardization of turnout material, a questionnaire was developed and sent to eight U.S. railroads and five manufacturers of turnout material. Replies were received from all requested. A summary of the results from the manufacturer is

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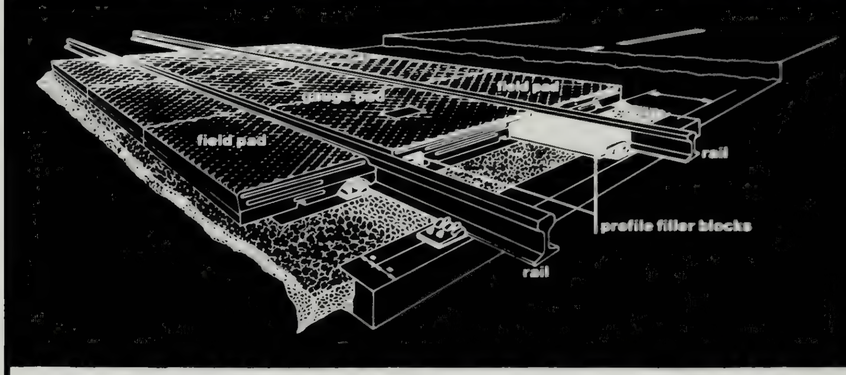

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shown in Table 2, and the railroad "users" in Table 3. An explanation of the questionnaire and the results as listed are shown in the following.

First, I must explain that in the questionnaire presented there were several standardization assumptions made to both the manufacturers and the railroads. Each was to assume, in their thinking, that a standardization of turnout sizes of 12, 16 and 20 would be produced and used; rail weight of 115 and 136 pound; switch points would all be Samson with manganese points; stock rails would be with undercut planing the full length; all frogs would be AREA heavy wall standard manganese insert with long frog arms for welding with two-hole drilling; guard rails would be all AREA standard bolted design.

These assumptions were made only to set the stage for both the using railroad and the manufacturers to think through the possible savings they might incur under such a standardization. They were in no way intended to postulate them as "the" standard. This would have to be developed in a joint railroad and manufacturers committee to ensure acceptance and economies.

Table 2

Manufacturer

Inventory		
	A. Raw material	12.2%
	B. Finished material	2.0%
	C. In process	4.4%
	D. Support material	7.0%
	Average Inventory Savings	6.4%
Manufacturing		
	A. Set up	13.2%
	B. Production	3.6%
	C. Storage	4.8%
	D. Handling	8.8%
	E. Planning	10.6%
	F. Assembly	6.6%
	G. Paperwork	9.4%
	H. Small parts	8.8%
	I. Parts supply	9.6%
	Average Manufacturing Savings	8.4%
Quality		
	A. Production	5.8%
	B. Inspection	4.8%
	C. Rework	5.8%
	D. Material return	4.6%
	E. Worker efficiency	7.0%
	Average Quality Savings	5.6%
Handling		
	A. Handling Finish product	3.0%
	B. Handling raw material	8.0%
	C. Packaging finished	3.0%
	D. Shipping finished	3.0%
	Average Handling Savings	4.3%

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Facility		
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	C. Material yard	10.6%
	D. Storage area	5.0%
	E. Office	0.0%
	F. Mfgr. support staff	1.0%
	G. Property	3.0%
	Average Facility Saving	3.1%
Equipment		
	A. Manufacturing	2.0%
	B. Handling	3.0%
	C. Assembly	2.0%
	D. Shipping	0.0%
	Average Equipment Savings	1.8%

Assuming the following percent contribution of each category of the manufacturing operation times the percent savings of that operation will equal the total estimated manufacturing savings.

	Contribution	**Est. Savings**	**Net Savings**
Inventory	10%	6.4%	0.64
Manufacturing	60%	8.4%	5.04
Quality	10%	5.6%	0.56
Handling	6%	4.3%	0.26
Facility	6%	3.1%	0.20
Equipment	8%	1.8%	0.14
		Total Est. Mfgr. Savings	***6.84%**

*Assume overhead and profit included in each category of manufacturing operation.

Manufacturer

The manufacturers were asked for their estimate of possible savings that could be made in inventory, manufacturing, quality, handling, facilities and equipment. Each ranked their estimate in percent saved for each operation. In an interview with one manufacturer's chief engineer each category was then ranked by its individual percentage to the cost of the overall manufacturer of the turnout material. This is shown at the bottom of Table 2.

In the inventory analysis, an overall estimated savings of 6.4% can be seen. The biggest projected savings was in the inventory of raw material.

In their analysis of savings in their manufacturing process, the suppliers estimated an 8.4% savings. The categories of setup and planning were leaders in their possible savings, followed closely by parts supply, paper work and handling.

The savings in quality of their production was estimated to be 5.6%. Worker efficiency was the leading predictor in the quality category.

Handling of material in both the raw material and finished product was predicted to save the manufacturer 4.3% of his cost. The reduced quantity of raw material was the leading category in these predicted savings.

In analyzing their savings that might occur in their facility needs, the manufacturer predicted a 3.1% savings. The material yard requirements was the leading contributor to these savings.

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In looking at their manufacturing equipment required under the projected standardization, a manufacturer predicted only a 1.8% savings. There were no special leading categories but just a general small estimated savings.

In summary of the manufacturers estimated savings, a percent of total manufacturing cost was placed on each of the questionnaire categories. This was done in conjunction with one manufacturer's chief engineer. Although these might vary from one manufacturer to another in the percentage breakdown, they were used to calculate the total manufacturer savings of turnout material produced. This produced an estimated 6.84% savings in the manufacture of turnout material.

Table 3
Railroad User (Average of 8 reporting railroads)

Inventory		
	A. Ordering	5.6%
	B. Maintenance	5.8%
	C. Emergency	6.3%
	Average estimated savings	5.9%
Quality		
	A. Material received	4.5%
	B. Installation	5.8%
	C. Inspection	6.5%
	D. Maintenance	4.5%
	Average estimated savings	5.3%
General (Estimated overall savings)		
	A. Overall material savings	7.0%
	B. Overall turnout savings	7.2%
	Average overall estimated savings	14.2%

Estimated savings to user would be the average of inventory and quality projected by the user questionnaire or 5.6%. The estimated overall savings of 11.2% is close to their overall prediction of 14.2%.

Railroad User

The eight railroads were asked for their estimate of possible savings they might incur in the standardization of turnout material in their inventory, quality of material received and a general overall prediction of savings. Each ranked their estimated savings in percentage of each category. In accumulating these statistics, the high and low rating was eliminated in each category. These results are shown in Table 3.

In their estimate of savings in inventory, the railroads predicted a 5.9% savings. The estimated savings seemed to be about the same across all of the listed categories of ordering, maintenance and emergency material inventories.

In analyzing their possible savings in the quality of the material purchased, the railroad estimated an average savings of 5.3%. The field inspection of material received was the leader in this estimate with material received, installation and maintenance material closely behind.

In requesting the railroads prediction of their estimate of the possible overall turnout savings from the postulated standardization, a check was sought to compare to the individual categories. Table 2 shows a reasonable agreement in the 11.2% versus the 14.2%. This study will use the 11.2% in further calculation as the railroad user prediction of savings from standardization.

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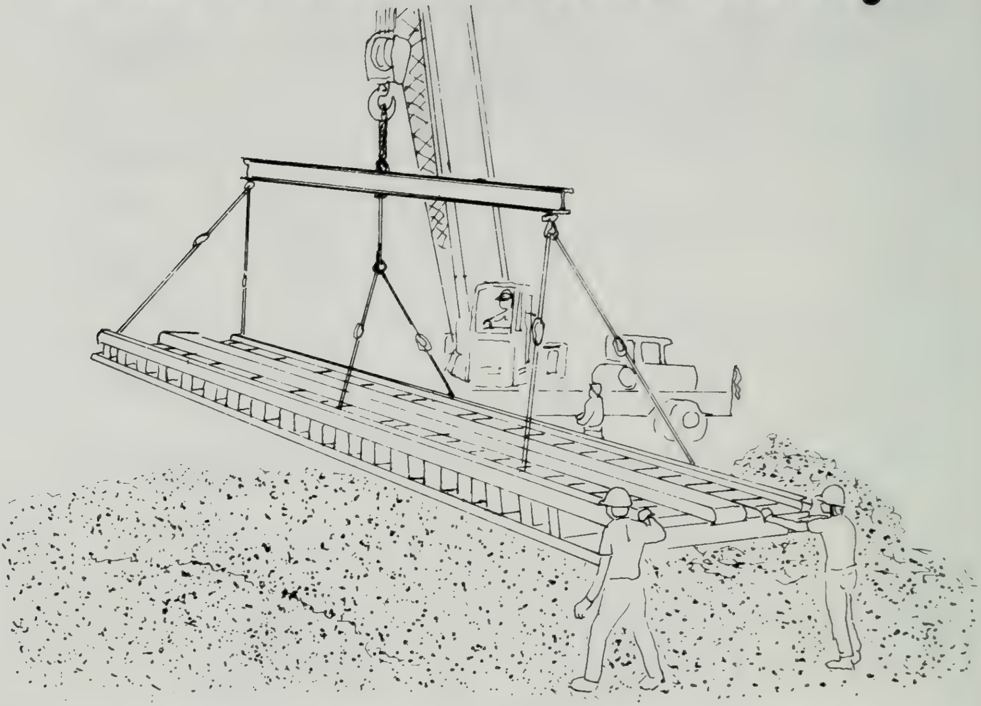

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Table 4

Assumption:		
Existing T.O.'s in B.L. or M.L. to be split by:		
No. 10/12	32,490	50% Low Tonnage 50% Med. Tonnage
No. 15/16	37,115	50% Med. Tonnage 50% High Tonnage
No. 20/24	13,379	100% High Tonnage
Summary Replacement Value (Millions):		
Low Tonnage		$409.4
Med. Tonnage		$484.3
High Tonnage		$256.1
Assume Life:		
Low Tonnage (1 to 10 MGT)		40 years
Med. Tonnage (10 to 20 MGT)		26 years
High Tonnage (over 20 MGT)		20 years

Calculation of Savings

In the railroaders questionnaire they were asked how many years they predicted it would take to effectively standardize turnout material. This prediction averaged 16 plus years. These estimates range from 5 to 35 years. Believing it could be no less than one complete mainline renewal cycle or turnout life as shown in Table 4. Thus, the life for effective complete savings for each traffic category will be used as shown.

Additional assumptions were made in the replacement of turnout life to be the separate categories of low, medium and high tonnages. It was also assumed that 50% of the number 10/12 turnouts would be low tonnage and 50% of the medium tonnage turnouts. Turnout sizes 15/16 would be 50% medium tonnage and 50% high tonnage lines. It was then assumed the 20/24 turnouts would be 100% high tonnage lines. These separations and totals are shown on Table 4.

The total replacement value is calculated using the percentage splits shown in Table 4 and in conjunction with the calculated total values of those splits shown on Table 1. The results of these calculations are shown on Table 4.

Also shown on Table 4 is the estimated replacement life of each of the categories of traffic.

In calculating the possible savings that might be derived by the industry in standardization, a few more assumptions were made. The assumption of an interest rate at 10% using a life renewal cycle year timeframe, shown in Table 4, for complete savings cycle we calculated the savings of:

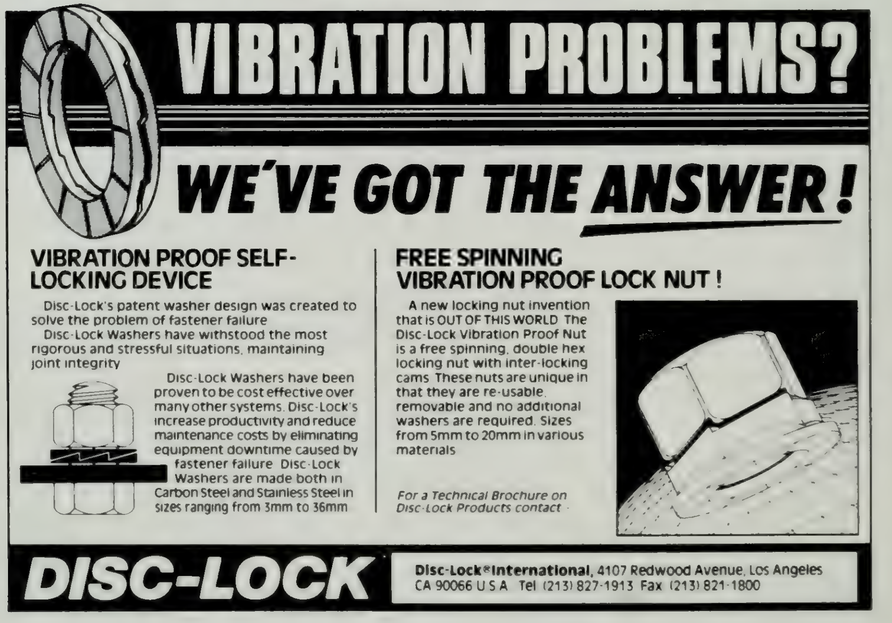
Low Tonnage	
Maintenance	32,490 T.O.'s × $506 × 18.04% = $2.97
Renewal	$409.4 M - 40 yr. × 18.04% = $1.8M
Medium Tonnage	
Maintenance	37,115 T.O.'s × $506 × 18.04% = $3.89
Renewal	$484.3M - 26 yr. × 18.04% = $0.3M
High Tonnage	
Maintenance	13,371 T.O.'s × $506 × 18.04% = $1.22
Renewal	$256.1M - 20 yr. × 18.04% = $0.2M
Total Annual Estimated Savings	**$10.4M**

Using the annual savings of $10.4 million and at 10% interest rate over a 20 year cycle, we see the net present value of $87 million.

Conclusion

In the investigation many assumptions were required to be made. They were necessary due to the magnitude of the sheer number of turnouts in the United States railroads as well as the difficulty in determining many of the details such as weight of rail, turnout size and specific design details within each. The assumptions on renewal cycles, annual maintenance, various tonnage category breakdowns and in general, each railroad's and manufacturer's overall approach in estimating savings makes the calculations made somewhat questionable.

Thus, in conclusion, it does not appear standardization of turnout material will provide the economics necessary to each individual railroad property to ensure joint movement towards standardization. The real problem is the magnitude of the turnouts in service in U.S. railroads are of small size on medium to low tonnage lines with very long renewal cycle times. Taking these small savings and adding the fact that inventories throughout the cycle time to complete standardization would increase, thus increasing costs which were not estimated, makes the case for standardization of turnout material not economically justified.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COMMITTEE 34—SCALES

Chairman: F. J. Loyd, Jr.

AAR CIRCULAR NO. OT-17-F* ON RECOMMENDED PRACTICES FOR INSPECTING, TESTING AND REPAIRING INDUSTRY TRACK, VEHICLE, HOPPER TYPE AND BELT CONVEYOR SCALES

Subcommittee Chairman: J. J. Robinson

The following rules, governing the inspection, testing and repairing of heavy capacity industry scales, are submitted as Recommended Practices:

National Code of Weighing Rules

The National Code of Rules Governing the Weighing and Reweighing of Carload Freight, endorsed by the Interstate Commerce Commission, June 9, 1914, provide that "When weights obtained on railroad or private scales are used for the assessment of freight charges such scales shall be maintained, tested and operated in accordance with the Track Scale Specifications and Rules approved by The American Railway Association (now Association of American Railroads)." Other types of commercial weighing devices, including vehicle scales, hopper type scales and belt conveyor scales are, when utilized for the assessment of freight charges, also subject to specific requirements. These scale specifications and rules are published in the AAR Scale Handbook and this pamphlet may be obtained from the Engineering Division, AAR, 50 F Street, N.W., Washington, D.C. 20001.

Design of Scales and Installation

The efficiency, ease and accuracy of adjustment and consistency of correct performance of a scale are dependent almost entirely upon the capacity, design and method of installation, and shippers should be urged to give the utmost consideration to these features when purchasing such scales.

Capacity

The capacity rating should be based on the AAR Scale Specifications and Rules referred to above.

NOTE 1—When, upon request to the proper officer, a special trip of a scale inspector or special movement of test cars is made, charges may be as provided in Rule 6, and an additional charge may be made for each test car moved or inspection made out of route or regular itinerary, except that when two or more parties request a special inspection and test, the additional charge for transportation will be proportioned equally.

NOTE 2—When the test of any of the classes of scales specified in Rule 2 requires movement of a test car by other lines than the testing carrier, any applicable freight and switching charges on such test equipment will be paid by the industry or railroad for whom the test is made.

Report

3. Immediately after the inspection and test of any individual scale has been performed by a scale inspector, and before leaving the premises, he should furnish the owner with a report summarizing the results of his inspection and test, specifically indicating any repairs and renewals required to put such scale in proper weighing condition. If possible, the scale owner or his representative should be present during the inspection and test.

Repairs

4. The service performed by the testing carrier when testing private scales may include such mechanical adjustments as are necessary, however, adjustments to electronic scale equipment should

---

*Circular No. OT-17-F was recently approved by the O-T General Committee, after revision and endorsement by Committee 34-Scales.

not normally be made by the testing carrier. Minor repairs may be made under the supervision of the scale inspector without causing delay to the regular movement of the testing equipment and the performance of the inspector's regular duties, but such service does not include the furnishing of material, the cleaning of scales or other extensive repairs.

5. If, upon inspection and test, the scale inspector determines that the strength of the scale is so restricted, or the condition of its foundations, lever systems, platforms, weighbridge supports, etc., is such as to render it impossible to adjust, or to maintain adjustment within the limits prescribed by the Scale Specifications and Rules approved by the Association of American Railroads, he will notify the proper railroad officials, who will take the necessary action to protect the interest of the carrier.

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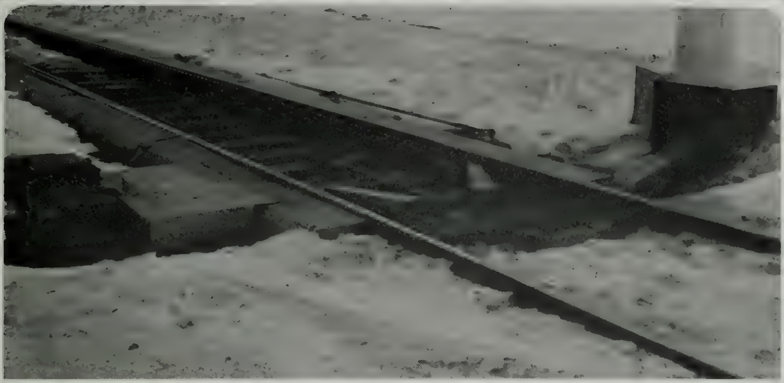

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AREA Short Line Committee—See Pg. 349

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Light density line in Maine

AREA Forms New Committee 18 — Light Density and Short Line Railways

The organizational meeting of AREA's newest committee was held on September 16th in Chicago, chaired by Jerry W. Heavin, Director of Quality on the Union Pacific, who was formerly President of the Red River Valley and Western Railroad. In conjunction with the meeting Robert E. Larsen, Director Industrial Development & Engineering, Green Bay and Western Railroad Company accepted the Vice Chairmanship of Committee 18, and Robert J. McCown, Program Manager for ENSCO, Inc. will be Secretary.

Committee 18 will be responsible for information and recommended practices regarding the special engineering, economic and maintenance needs of light density and short line railways. Subcommittees were established as follows:

> Subcommittee B will review the current Manual for its application to shortlines, including previously deleted portions that may be valuable. It will be chaired by Michael G. Arter, V.P. and Chief Engineer of the Dakota Minnesota & Eastern Railroad.
>
> Subcommittee C-1-90 will investigate used track material. The subcommittee is planning to develop an inventory listing of rail less than 100#. It will also recommend practices for obtaining, maintaining, organizing and operating maintenance of way work equipment on shortlines and will be chaired by Carl T. Broussard, Asst. Gen. Mgr. - Maintenance, Louisiana and Delta Railroad, Inc.
>
> Subcommittee D-1-90 will investigate contracted engineering, repair, and maintenance services for short lines. They will also investigate inspection criteria.

It is expected that there will be much interaction between Committee 18 and the other 24 AREA Committees in accomplishing these as well as future subcommittee tasks.

The first regular meeting of the committee will be in connection with the Annual Technical Conference of the AREA in Chicago, March 17-20, 1991. Individuals interested in joining the committee may call or write Tom Smithberger at AREA Headquarters, Room 7702, 50 'F' St., N.W., Washington, D.C. 20001, (202) 639-2190.

Committee 18
Chairman J. W. Heavin

TRACK SURFACING WITH CONVENTIONAL TAMPING AND STONE INJECTION**
(abridged AAR Report R-719)

By: S. M. Chrismer*

1.0 INTRODUCTION

With so much emphasis on productivity, the railroad maintenance officer may be missing another important aspect of track maintenance; that is, how long-lasting are the effects of a certain procedure. The effectiveness of maintenance must be measured not only by the quality of the job just after treatment, but also by the durability of that maintenance.

The report focuses on one aspect of track maintenance; track surfacing. The effectiveness of tamping is appraised and compared with a relatively new alternative track smoothing procedure known as stone injection. The report describes the process of stone injection and how the track performs with this maintenance relative to tamping. Also discussed is the process of tamping. Although tamping is thought of as a well understood technique, there are some aspects of it which are not widely known.

Although British Rail (BR) found that injected track lasted three or more times as long (compared to tamped track) before returning to it's pre-maintenance profile roughness [1], the injection method had to be proven under the heavier axle loads prevalent in the U.S. Results of comparison tests of the two techniques on test track and revenue track are shown. The performance of the two methods is compared on the basis of of durability of smoothed track profile.

Finally, conclusions are offered based on the research findings.

2.0 WHAT IS STONE INJECTION?

Stone Injection is the process of surfacing by raising certain "low" ties and placing a measured amount of half-inch to three-quarter inch stone under them by blowing the stones through a tube. Though the general practice dates back to what was known in the 1930's as "measured shovel packing" or trowling, the tools used to place the stone have been under considerable development. There is now a mechanized version of stone injection which is being used on British Rail. Because the stone injector machine uses the same principles as the hand-held method used in this test, there should be no difference in performance of track which is maintained by either version. The following describes the stone injection process using the manual method.

Before injecting stone, a pre-injection top-of-rail survey is performed to indicate the amount of stone each tie should receive to bring the rail to a level condition. The survey instrument could be an optical level. In this study, a unit known as a FROG was used to survey the pre-maintenance profile. The FROG (see Exhibit 1) is a four-wheel dolly which uses slope inclinometers to measure the slope between the front and back right-hand wheels to give profile, and the slope between front right and left wheels to give cross-level. The result, when reduced by a computer program, is the unloaded top-of-rail (TOR) profiles for both rails. The FROG can also be used after maintenance to determine the changes in track profile as traffic tonnage accumulates.

The amount of stone to be injected under the tie under each rail depends on the amount of raise desired. The relationship between weight of half-inch stone to height of raise of one rail is one pound for every thirty-second of an inch. Two injectors are used at each rail seat, one on each side of the rail. Each injector will receive half of the predetermined amount of stone.

Because certain ties will have larger than average voids between them and the ballast bed, the above relationship may underestimate the amount of stone needed since it is based on an unloaded TOR survey. To measure this gap a voidmeter is used, as shown in Exhibit 2. It fits on the base of the rail in the ballast crib. The plastic spacer is pushed tight against the seat. As traffic passes over, the plunger moves down and up under each axle load pushing the spacer down the furthest under the maximum axle load. After the traffic passes, a "feeler" gauge is inserted between the spacer and the main bracket to

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*Senior Research Engineer, Association of American Railroads

**Presentation to the A.R.E.A. Fall Technical Conference, Jacksonville, Florida, October 25, 1990

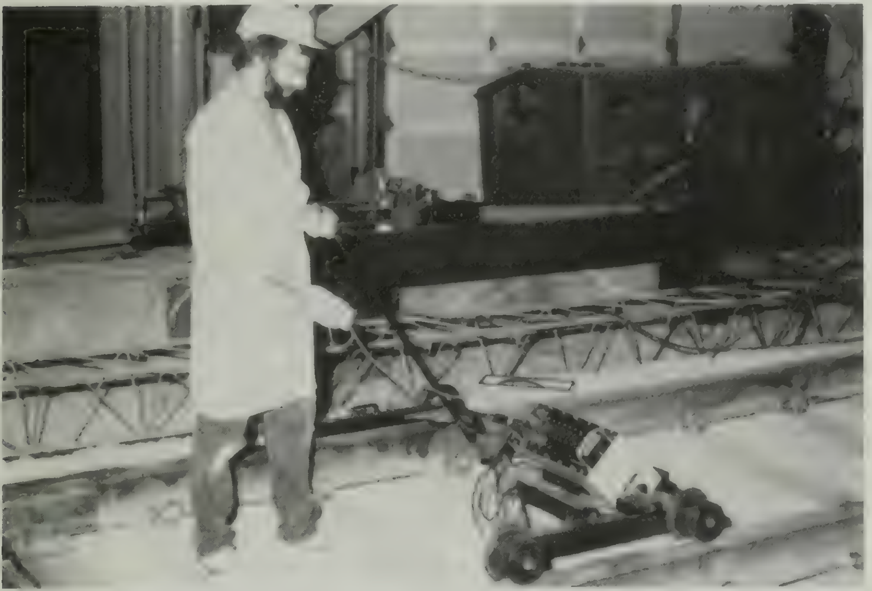


Exhibit 1. FROG Device Used to Measure Track Profile.

determine the depth of the void. If the void is is greater than one-quarter of an inch, an additional pound of stone must be placed for every thirty-second of an inch the gap is greater than one-quarter inch. A one-quarter inch gap is considered typical so any void less than this needs no additional stone.

Exhibit 2. Void Meter to Measure Gap Between Ballast and Tie.

With this preliminary information a few track workers and a transit and rod man are required at the site. The survey team will first take readings of the elevations of the tie ends. This will allow the surveyor to determine when the track is lifted high enough to create the desired two inch gap under the tie to allow a path for the stone. Taking the level readings from the *tie ends* is important for wood ties with cut spike fasteners since these will not always hold the tie plate tight against the rail base during lifting of the rail. For ties with direct fixation fasteners, all levels may be taken from top-of-rail since there will probably be no appreciable gap between the tie and rail. Once the elevations for all ties to be lifted are known, the surveyor adds two inches to each reading. This new level is the elevation to which the tie must be lifted by jacking to allow a two-inch gap under the tie.

Jacks are placed under the rails in the cribs at the start of the section. The type of jack should be the "flat" or "pancake" with telescoping sections. This type has the low height needed to be placed under the rail. It may also be left in place (although with the pressure released) when traffic passes over. Even with this type of jack it may be necessary to push some crib ballast away from under the rail.

At the first tie to receive some stone the surveying rod is placed on either end of the tie and the surveyor tells the trackman how much to pump the jack until the tie is at the desired elevation. This process progresses by moving the jack two or three cribs away after the injectors are placed. Having more than two jacks greatly speeds up the work by allowing the surveyors to proceed well ahead of the injection.

An impact hammer (using air from the compressor) drives the injector tubes into the crib ballast along one side of the tie at the locations in Exhibit 3. Only one side of the tube has an opening; that side faces the tie. The injector is driven only so far so as to expose a slot height of about two inches from the tube invert to the bottom of the tie. In this way the tube invert should be at the same elevation as the

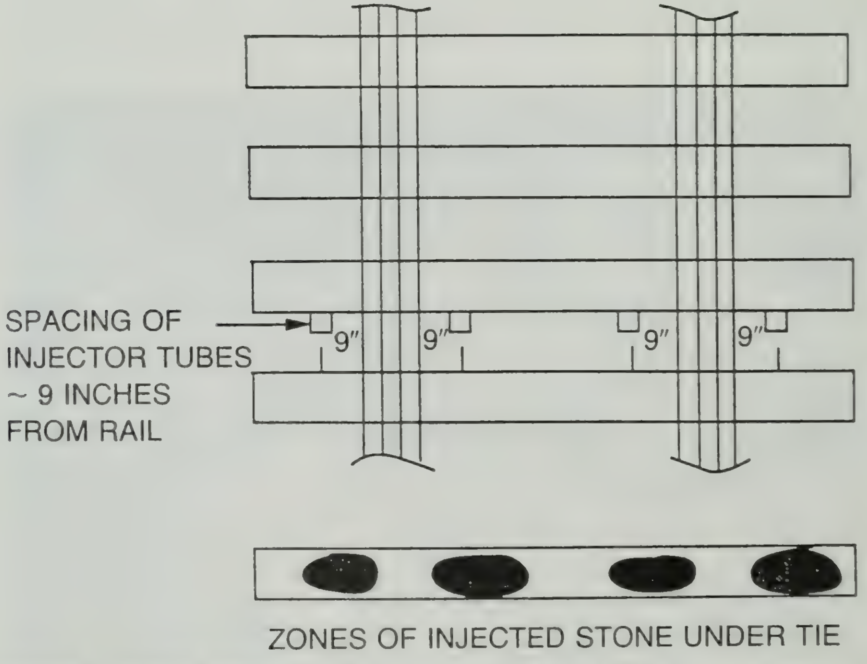

SPACING OF INJECTOR TUBES ~ 9 INCHES FROM RAIL
9"
9"
9"
9"
ZONES OF INJECTED STONE UNDER TIE

Exhibit 3. Location of Injectors Along Tie and Pattern of Stone Under Tie After Injection.

ballast bed. This height of tie raise is high enough to allow the stone to be placed but small enough to prevent a significant amount of crib ballast from falling in. The operator drives the tubes snug against the tie until markings on the outside of the tube line up with the top of the tie.

The hand-held injectors are attacked to an air compressor. The air enters the tube through an eighth-inch opening which allows the air to attain a high velocity. It is this velocity of air, not the air pressure, which forces the stone under the tie. As a weighed amount of stone is poured into the tube hopper, the stone falls by gravity into the tube and enters the air stream. The high velocity air accelerates the particles with sufficient energy to blow them under the tie into the formation shown in Exhibit 3. Examinations have shown each area of support to be about nine inches by 18 inches; about the same area the tamping tines affect.

The half-inch stone does not percolate down through the existing ballast bed because voids in typical ballast gradations are significantly smaller than 0.5 inch. BR has found there to be no percolation after several years. Nor does the stone "foul" the ballast. Half-inch stone is present in limited quantities in most gradations and the small amount being placed under tie will not significantly affect the ballast gradation. The stone may even help by lifting the tie out of a fouled, wet ballast.

2.1 Effect of Stone Injection on Track Profile

Assume one is to maintain the hypothetical track shown in Exhibit 4. The profile is a combination of small and large dips that must be raised. While tamping would likely raise the track over the entire track section, stone injection technique would be applied to only those ties in the locations of low profile.

After injection the raised profile in Exhibit 4 will result. Note that where the sections were dipped, they are now raised. This situation, where the "mirror image" profile is produced, is only temporary, for within the passing of a few trains the profile would settle down. Thereafter the track would slowly settle with MGT accumulation as the track actually improves in smoothness. After the accumulation of

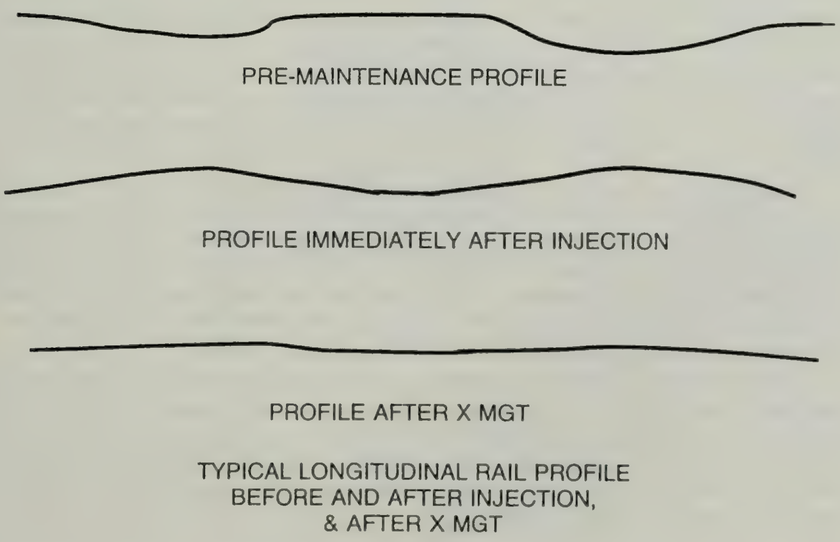

PRE-MAINTENANCE PROFILE
PROFILE IMMEDIATELY AFTER INJECTION
PROFILE AFTER X MGT
TYPICAL LONGITUDINAL RAIL PROFILE
BEFORE AND AFTER INJECTION,
& AFTER X MGT

Exhibit 4. Typical Track Profiles Before and After Injection.

MGT the track will eventually return to its pre-maintenance degree of roughness. BR has found that stone injected track takes about at least three times longer to do this compared to tamped track.

The main reason for this longer life appears to be in the nature of the two techniques. Typical depths of stone placed under the tie range from a fraction of an inch to just over one inch. This situation of having an inch or so of stone over a compacted, undisturbed bed of ballast produces a more stable track bed than does tamping. Whereas injection adds an inch or so of stone to a stable base, tamping disturbs the ballast to a depth of six or seven inches. The resettlement of this depth of ballast as traffic compacts it is what gives tamping its characteristically short maintenance life.

The large magnitude and repetitive nature of traffic loading controls ballast compaction, and so, ballast strength. Therefore it is better to leave the ballast under the tie undisturbed and add small amounts of stone to adjust rail level.

3.0 EFFECTS OF TAMPING

The word "tamping" is something of a misnomer as it implies compaction of the ballast due to tamping action. Compaction may occur under some circumstances but tamping has been found to often result in a loss of compaction. Selig has determined that ballast density and bearing strength decreased significantly after tamping [1].

3.1 Tamper-Caused Ballast Degradation

A test was performed at the Transportation Test Center (TTC) at Pueblo to determine how much ballast damage is caused by tamping. Two ballast materials, a granite and a limestone, were first graded so that no material passed the quarter-inch sieve and so that the initial gradation was known. To contain the ballast, track steel boxes were constructed and placed under ties as shown in Exhibit 5. Each tie had two boxes underneath; one for each rail. The boxes had four sides but no top or bottom. The pre-graded ballast was placed in each box. The granite gradation was within the range designated as AREA #4 while the limestone was AREA #3.

These two ballast materials were used to determine the effect of ballast quality on the amount of tamping damage. The granite had a Los Angeles Abrasion loss of 24% and the limestone had a loss of 38% using the ASTM C535 procedure. The ballast was tamped 10 times in one set of boxes and 20 times in the other. The results, shown in Exhibit 6, relate to the amount of 1/4″ material (fines) which resulted from the tamping. Degradation appears to increase slightly non-linearly with the number of tamping squeezes. As a general rule, it appears that tamping produces one to two pounds of fines per squeeze.

A more useful way of looking at the degradation would be to relate it to the amount of reduction of ballast life. But ballast life is largely controlled by the ballast *under the tie*, while the ballast gradation before and after tamping was performed on the *entire* bin of ballast in this test. Because it was noted from visual observation that virtually all the ballast damage occurred in the tamping zone (Exhibit 5), it appears that the breakdown measured in the sieve analysis after tamping occured in this volume. Therefore, to isolate the breakdown to only the ballast in this zone, an estimate was made of the percent of ballast by weight in the tamping zone of the bin.

To find the decrease in ballast life under the tie, the change in gradation of ballast in this volume is estimated by first finding the volume shaded in Exhibit 5. This was calculated as 2.6 cubic feet. Assuming a ballast compacted unit weight of 120 pcf (the density achieved under typical tamping conditions) this gives a weight of material in this zone of about 312 pounds. With this estimate, it is then possible to "back-calculate" the percent material passing the 1/4″ sieve knowing the total weight of material that passed this sieve from Exhibit 6.

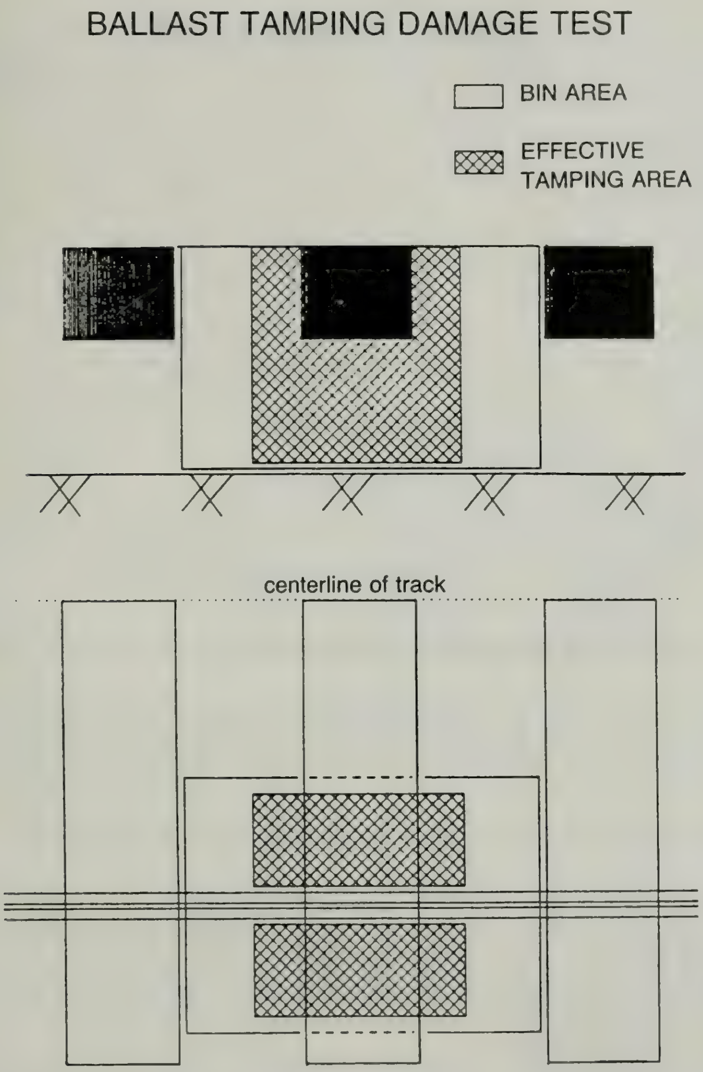


Exhibit 5. Volume of Ballast Affect by Tamping.

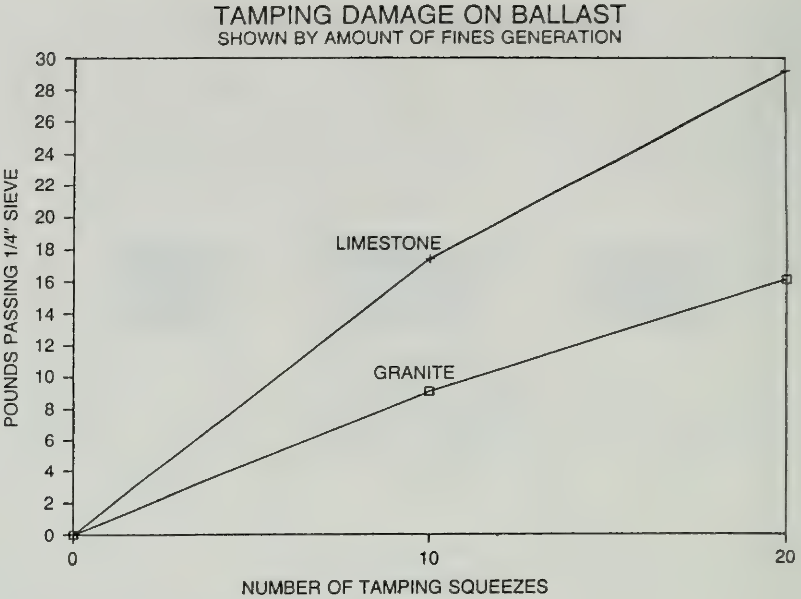


Exhibit 6. Amount of Fines Generated by Ballast Tamping

Exhibits 7 and 8 represent the estimated gradation of ballast in the tamped zone. The amount of material passing No. 4 sieve is often used to judge ballast life with 20% passing considered the upper limit. It is seen that the worst case, that of limestone ballast tamped 20 times, destroys about 40% of the ballast life.

4.0 COMPARISON OF CONVENTIONAL TAMPING AND STONE INJECTION

4.1 Test on Transportation Test Center Track

Prior to any tests on revenue track it was decided to try a comparison test of tamping and stone injection on test track. This also allowed a better understanding of the technique of stone injection. The results of TOR profiles taken at several intervals up to 50 MGT are shown in Exhibit 9. Note that the tamped section returned to its pre-maintenance profile within 50 MGT while the injected track was still improving in profile after 50 MGT.

4.2 Test on Norfolk Southern

In August, 1987, two crossings outside Petersburg, Virginia were treated with tamping and stone injection. The track carries 48 MGT per year which is mostly Eastbound coal traffic. Because the traffic is in one direction it was decided to perform stone injection on the west side of crossing #1 and tamp on the east side, then to reverse this for crossing #2. In this way, the effects of traffic direction would be cancelled.

The test zones extended from the crossing to 150 ties away on either side of the crossing. Not all of the ties on the stone injection side of the crossing received lifting and placement of stone. Remember that ties receive stone only where the rail is low. On the tamped side of the crossing the tamper treated the full 150 tie section, running out the tamping lift to zero at the end of the test section.

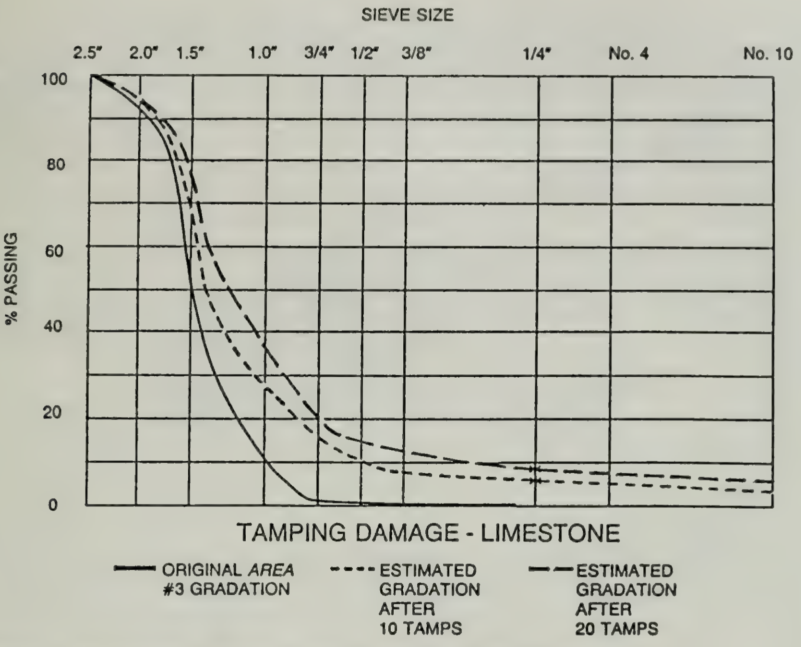


Exhibit 7. Gradation of Limestone Ballast Before and After Tamping.

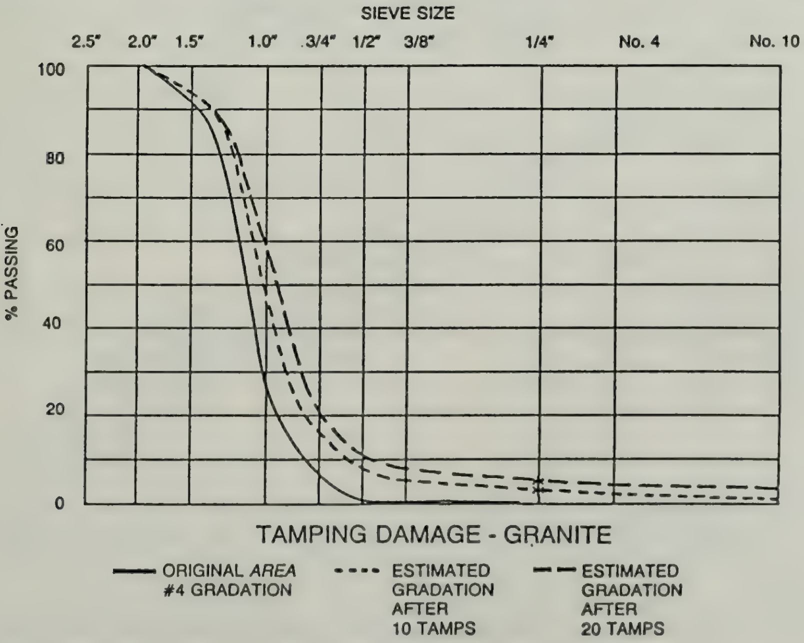
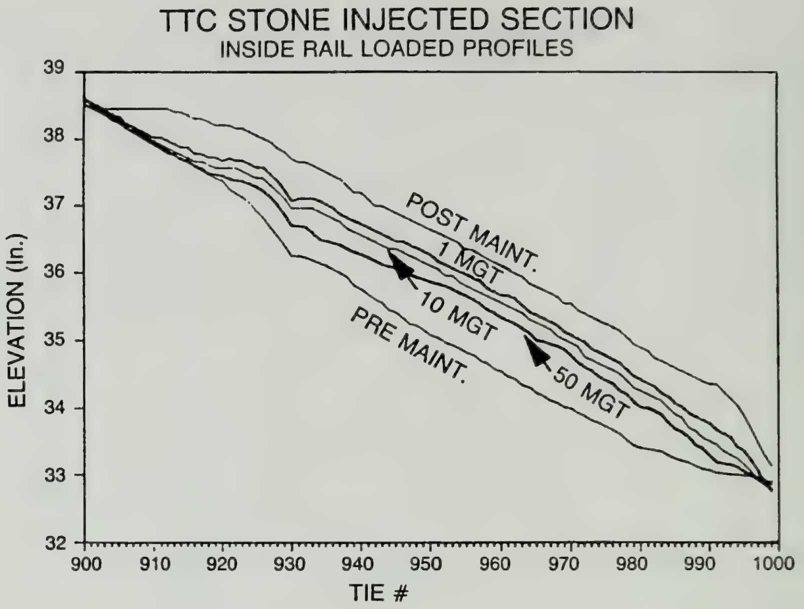


Exhibit 8. Gradation of Granite Ballast Before and After Tamping.



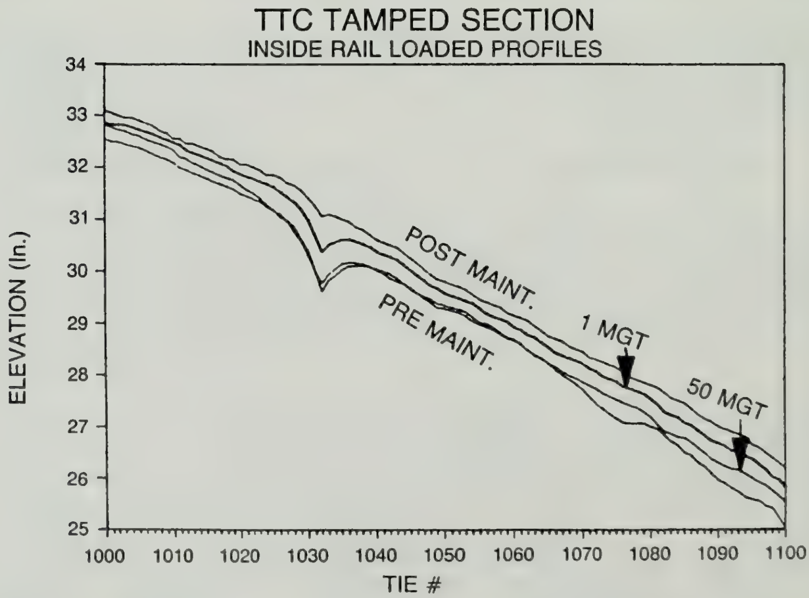


Exhibit 9. Track Profile at Transportation Test Center Before and After Stone Injection and Tamping.

A unit known as a FROG (as discussed earlier) was used to measure track profile before and after maintenance. Subsequent measurements have been taken at 10, 50, and 80 MGT as well. Before maintenance occurred there were distinct dips in rail level on either side of the crossings. Both maintenance methods were successful in removing these dips at both crossings. The difference in the two methods is shown by track performance after traffic started to pass over. Within 10 MGT the tamped sides lost most of their lift and smoothness. By the time 50 MGT is accumulated, the tamped sides virtually returned to their pre-maintenance profile and degree of roughness. Therefore, for these sites, the "life" of the tamped sections is about 50 MGT.

As the injected sides of the crossing have settled under traffic the surface has become smoother with traffic. The "life" of the injected track appears to be substantially longer than 50 MGT based on the graphs of standard deviation shown in Exhibits 10 and 11. Standard deviation is simply a measure of track roughness; the greater the value, the rougher the track.

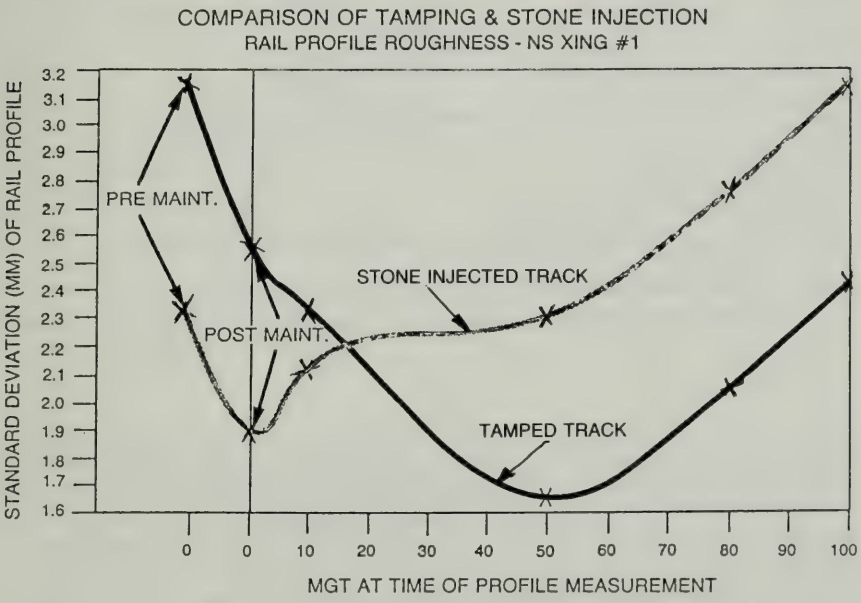


Exhibit 10. Track Profile Roughness of Norfolk Southern Crossing #1 Before and After Stone Injection and Tamping.

4.3 Test on the Burlington Northern

Stone injection and tamping were also performed on the BN single track mainline near Alliance, Nebraska. The track carries 120 MGT per year which is predominantly eastbound coal traffic. This test was performed on a track with no crossings or other items of fixed elevations nearby. Therefore, the track could be given a larger lift during conventional tamping than in the case of the Norfolk Southern tests where the crossings limited the allowable track raise. Also, the track faults were much longer, requiring 125 ties to be stone injected, and about 250 ties to be tamped.

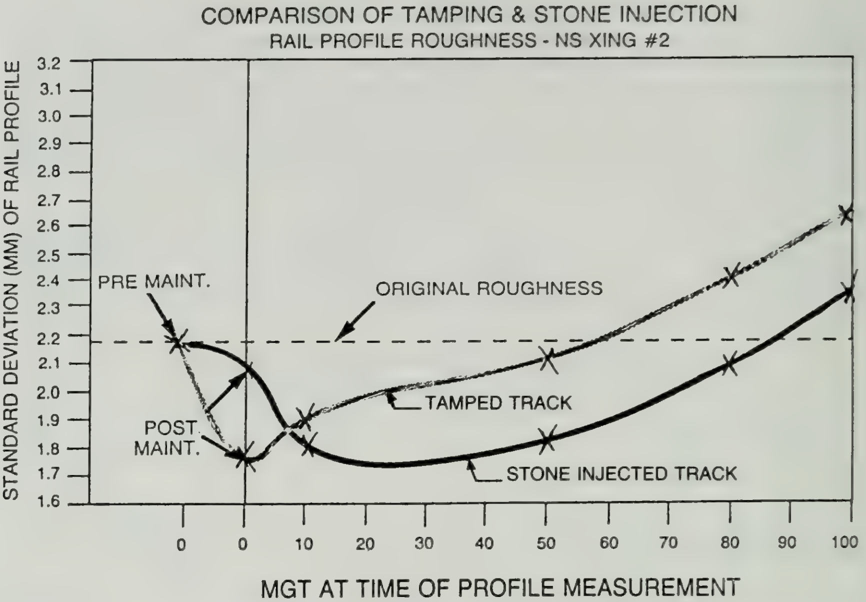


Exhibit 11. Track Profile Roughness of Norfolk Southern Crossing #2 Before and After Stone Injection and Tamping.

The injected track lost about the same amount of lift as the tamped track after 50 MGT. In terms of track profile roughness (see Exhibit 12), the stone injected track maintained its smoothed profile somewhat better than the tamped track. Unfortunately, the survey taken at the 50 MGT level was over every other tie rather than every tie so a calculation of standard deviation for 50 MGT is not possible. But by observing the trend of the data for the tamped track in Exhibit 12 it is possible to predict that it probably had returned to its pre-tamp roughness, or surpassed it, when 50 MGT had accumulated.

The AAR investigators were initially surprised that the tamped track maintained its track raise so well (if not its smoothed profile) as traffic accumulated. BR personnel indicated that they have observed tamping to produce this effect when the lift is greater than the approximate mid-size of the ballast particle sizes (the so-called D50 particle size). Lifts below such a height can quickly return to the pre-tamp elevation, while greater lifts allow more of a "permanent" raise. In these tests, considering the ballast gradations used by the BN and NS railroads, the D50 size is about one inch. Therefore, it is interesting to note that on those sites where the tamping lift is less than one inch (at TTC and on the NS railroad) the track did return quickly to its pre-tamp elevation. Where the lift is significantly greater (on the BN railroad), the lift is more "permanent."

4.4 Productivity Versus Durability

To weigh the economics of the choice between the two mechanized versions of track surfacing; tamping and injection, it is necessary to consider the greater productivity of tamping (in terms of ties per shift) versus the more durable results of injection. Mechanized injection is not as fast as conventional tamping. But an economic analysis will show that a tradeoff exists between the

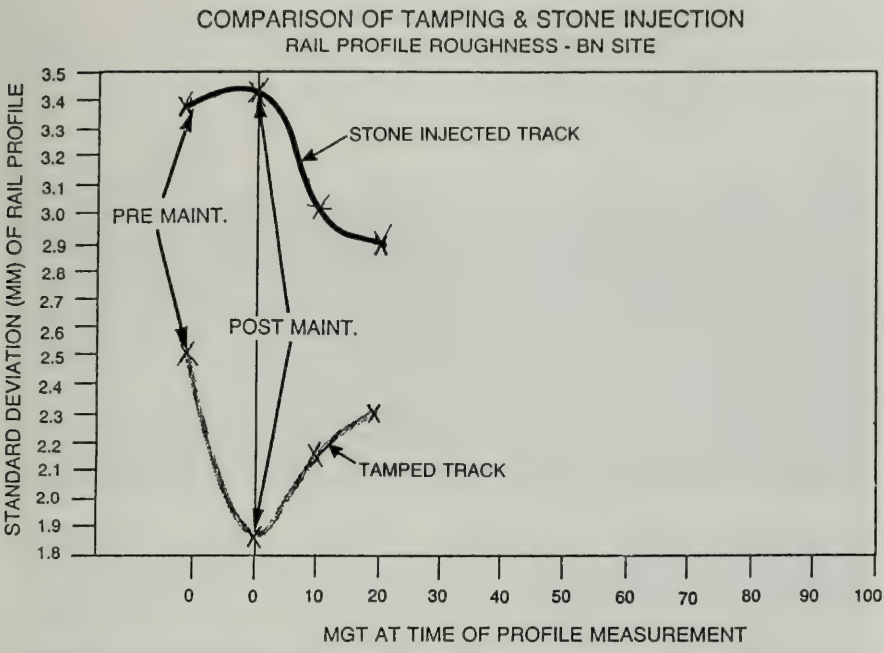


Exhibit 12. Track Profile Roughness of Burlington Northern Track Before and After Stone Injection and Tamping.

lengthened maintenance cycles of stone injection and the machine productivity of tampers. Also, because injection requires that only some of the ties have stone blown under them, and then only with a small amount of stone required per tie compared to tamping, the drastically reduced amount of ballast needed could significantly reduce the cost of track maintenance.

4.5 Other Considerations and Comparisons

As with tamping, a slow order is appropriate after stone injection. BR has found the lateral stability of track just after stone injection not to be significantly different from post-tamp lateral stability. The track will settle down with the passing of just a few trains. Therefore, the slow order issued would not need to be any longer in the case of injection.

Another BR finding concerning stone injection and tamping is that both appear to perform equally well where a high track raise would be allowed or required [4]. If clearances or fixed elevations are not a problem and the track can be tamped up two inches or more, then tamping appears to provide a surfacing life about as durable as stone injection. It seems that a sufficient height of track raise would allow enough ballast to be placed under the tie, allowing for a denser, rather than looser, ballast condition after tamping.

Finally, rough track is known to produce impact loads which can shorten ballast life as well other track component life. Therefore, track which settles more quickly and returns to its pre-maintenance roughness will just as quickly allow these dynamic wheel-rail impact forces to develop. Exhibit 13 shows an example of this problem, where a track with several surface dips produced a harmonic response in the car suspension leading to a peak wheel load on a test vehicle of 82 kips [2]. The static wheel load was 25 kips.

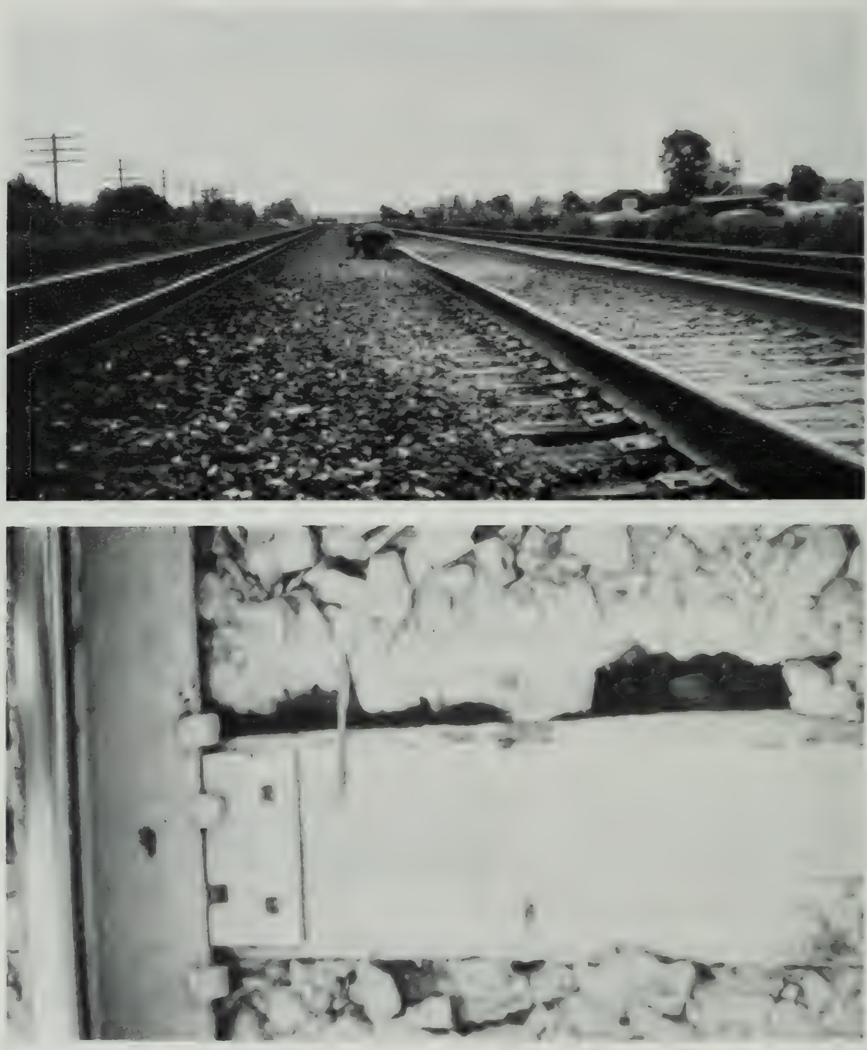


Exhibit 13. Photos of a Multiple Track Irregularity Site, Mud Pumping Open Track.

Of course, rail replacement, grinding, or straightening may be a better solution for "short wavelength" track faults where bent rail or bad welds are the cause of poor track surface. Although stone injection is at least as suitable as tamping for such conditions, neither stone injection or tamping is meant to address such problems since there is little that ballast-related maintenance can do for these "short wavelength" irregularities. The ballast will merely be pushed back down in short time. But for those longer wavelength track faults of ten feet or more, the longer maintenance life provided by stone injection should reduce maintenance costs.

5.0 Conclusions

Based on the findings of the field tests, the following conclusions are offered.

1. Judging by the trends of track profile roughness with MGT, stone injection lengthened the maintenance cycle by a factor of at least two compared to tamping by providing a more durable smoothing effect on track.

2. Track which was tamped with "skin lifts" (less than one inch) was observed to quickly return to its pre-tamp elevation and roughness. Tamping with lifts greater than about one inch seemed to provide a much more durable lift. BR has found that tamping raises should be greater than the mid-sized ballast particle size (about one inch for AREA #4 gradation) for the lift to be "stable."

3. Although stone injection was demonstrated to produce a more durable smoothed track in every case, it out-performed tamping the most when the allowable tamping raise was small. Therefore, stone injection may have its prime advantage in areas of fixed elevations such as bridge approaches or grade crossings where large raises are not tolerable.

4. Because stone injection is far less destructive to ballast, ballast life is not significantly decreased, unlike tamping which measureably decreases ballast life.

5. Track which returns more slowly to a rough profile imparts less dynamic load into the track and vehicles over time.

6. Tamping is faster than mechanized stone injection using current technologies. But there is an economic tradeoff between speed and durability of maintenance. If injection reduces maintenance cycles significantly, stone injection could be more economical for a given situation.

6.0 REFERENCES

1. Johnson, D. M., *"The Introduction of British Rail's Stoneblowing Technique as an Alternative to Tamping,"* paper presented at the 1983 AREA Conference, Chicago, March, 1983.

2. Selig, E. T., Yoo, T. S., Panuccio, C. M., *"Mechanics of Ballast Compaction, Volume 3: Field Test Results for Ballast Physical State Measurement,"* FRA/ORD-81/16.3, Federal Railroad Administration, Office of Research and Development, Washington, D.C., March, 1982

3. Kalaycioglu, S., and Tajaddini, A., *"Locating Vertical Track Irregularities Which Cause Excessive Vehicle Loads,"* AAR Report No. R-694, March, 1988.

4. Personal communication with D. M. Johnson, 1987.

INCREASING THE CAPACITY OF BRIDGE TRUSS TENSION MEMBERS VIA POST TENSIONING STRANDS**

By: Joseph J. Pullaro*

INTRODUCTION

By the addition of a relatively few number of high strength strands, mostly hidden from view, an economical method for increasing the capacity of truss tension members can be achieved. The method allows for structural upgrading of trusses to meet modern load criteria.

Photo 1. Walnut Street Truss Bridge in Chattanooga, Tennessee.

The Walnut Street Bridge crossing the Tennessee River in the City of Chattanooga, Tennessee is a pin connected Camelback Pratt Truss and has the distinction of being the oldest surviving bridge across the 1500 mi. river. The bridge was designed by a noted consulting engineer, Edwin Thatcher of Louisville, Kentucky, with construction completed in 1891. The structure replaced ferry service and helped unite both banks of the river during the time of great economic growth in the city. The bridge is 2370 ft. in length, with a 780 ft. iron viaduct forming one approach. Six (6) Pratt truss spans cross the river channel, three (3) spans at 207 ft. and three (3) spans at 317 ft. The trusses range from 28 ft. to 48 ft. deep. The roadway width is 18 ft., with two (2) 5 ft. sidewalks provided. After the construction of the nearby Market Street Bridge in 1917 the bridge saw a decrease in usage. The structure was rehabilitated on various occasions including the replacement of the timber superstructure with steel stringers and an asphalt wearing surface. In 1974, in an attempt to strengthen certain eye bars, U-shaped steel bands were wrapped around the heads. The bridge was eventually closed to all traffic in 1978 due to concerns about its structural integrity.

In the 1980's there began a community spirit to rehabilitiate the structure to serve as a pedestrian bridge and linear park. Contributions were raised from private citizen groups and together with a grant from the Federal government and funds from the City, the project became a reality. It is expected the rehabilitated structure will be extensively used for festivals, exhibits, etc. and be a focal point for the resurgence of the river front. In the future the bridge will also be used as a crossing for a trolley system planned by the City. Extreme care in restoring as many historic details as possible would be required as the bridge is of significant historic value and is eligible for inclusion on the National Register of Historic Places.

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The task of restoring the bridge includes many details including pier, deck and railing work. The major work however and the focus of this paper is to describe the method selected for strengthening the truss members, in particular the tension chords.

Steel eye bars make up the bottom chord and tension diagonals. The bars are in pairs and are up to 6″ x 1-1/8″ in section. A maximum of four (4) bars are used in the lower chord. The compression members including the top chord and verticals are composed of steel shapes (either angles or Z's). Existing information indicated low carbon steel was used for all tension members with an allowable design load of 16,000 psi. There was doubt, however, whether the eye bars were actually steel as indicated or wrought iron as the lack of severe rusting resembled that of a wrought iron member.

INSPECTION AND TESTING

An in-depth inspection of the trusses showed losses due to rusting and pitting at the eye bar heads to be a common condition. The inspection also revealed the presence of numerous dimples or concave depressions of unknown origin in the head and neck of the eye bars at a large number of the lower joints. No significant losses were noted in the main body of the eye bars. Ultrasonic investigation of the eye bars did not reveal the presence of any cracks.

Field metallography and chemical analysis confirmed the presence of low carbon steel. Metallography proved to be a conclusive test. This is an examination in which the surface of the eye bar is polished and etched and then the microstructure recorded on a plastic replica so that it can be examined in the laboratory. The replica clearly indicated microstructure of steel as it showed no evidence of the massive iron-silicate stringers that is a distinctive feature of wrought iron.

The structure was strain gauged utilizing an actual truck and results compared to theoretical values. Measured strains in the tension members were compared to computed values with the results considered consistent with actual behavior of trusses of this age.

TRUSS STRENGTHENING

The maximum computed stress in the eye bars under existing dead load is 11.9 ksi. In light of the condition at the eye bar heads it was decided that the maximum allowable stress in the eye bars should be limited to the existing levels, which the trusses have safely sustained. It was also considered prudent to build in redundancy into the truss tension members since many eye bars have existing flaws that may initiate cracks in a fracture critical member.

An analysis was performed on the eye bars to determine the remaining fatigue life. By obtaining past historical loading data and utilizing AASHTO methods it was determined that no more than 73% of the fatigue life was used. Highly conservative traffic assumptions were used in the analysis especially for the first 50 years of the bridge's life when reliable truck data was not available. Truck traffic for that period was set at recent levels, an admittedly conservative approach. It was therefore concluded then that fatigue will not be a concern for the anticipated loading of predominantly pedestrians and trolleys.

Conventional strengthening of the truss members by adding reinforcing members was first investigated. Due to the large number of members to be reinforced and the difficulty in providing attachments to the closely packed members, this alternate was rejected. Additionally the final stresses in the eye bars would certainly be higher than the current level as the new and old members would share the applied loads.

A system of post-tensioning the trusses was then explored. The possibility of relieving the eye bars of a large portion of their existing dead load would free up capacity needed to support future live loading. In addition post-tensioning would introduce redundancy in the tension members by providing an alternate load path via the strands.

A system of straight and deflected strands was selected for the post-tensioning. The strands were placed to coincide with the bottom chords and the diagonals fanning away from the mid-span. The strands were deflected at certain lower joints by wrapping them over specially constructed saddles attached to the lower joint pins. Dead anchors secured the strands to the bearing pins and the upper joint pins. See Figure #1. Tensioning of the strands is to be performed at the lower chord level via jacking station assemblies which are to be left in place should any adjustments be required in the future.

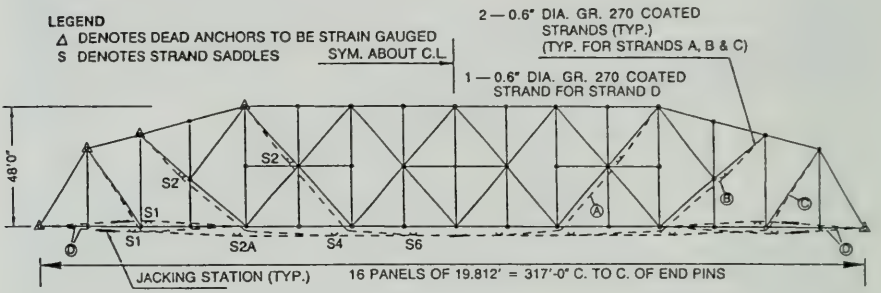
LEGEND
Δ DENOTES DEAD ANCHORS TO BE STRAIN GAUGED
S DENOTES STRAND SADDLES
SYM. ABOUT C.L.
2 — 0.6" DIA. GR. 270 COATED STRANDS (TYP.)
(TYP. FOR STRANDS A, B & C)
1 — 0.6" DIA. GR. 270 COATED STRAND FOR STRAND D
48'-0"
S2
S2
S1
S1
S2A
S4
S6
Ⓐ
Ⓑ
Ⓒ
Ⓓ
Ⓓ
JACKING STATION (TYP.)
16 PANELS OF 19.812' = 317'-0" C. TO C. OF END PINS

FIGURE #1 - ELEVATION

The strands are 0.6 in. diameter Grade 270 coated prestressing strands and are installed in pairs. The post-tensioning load introduced into the strands is 15 kips or less and results in an equal reduction of force in the eye bars. The strands constitute an internal system of post-tensioning and do not affect the forces in members that are not in line with the strands. The live load stress due to pedestrian loading will be shared by the strands and the eye bars in proportion to their stiffnesses. The eye bars being much stiffer than the strands will pick up most of the live load stress. Under full live load the eye bars will experience stress no greater than 10.9 ksi. Under proposed dead load the stress in the eye bars is reduced to less than 5 ksi, the remainder of the dead load being in the cables.

TABLE 1 - SUMMARY OF STRESSES

317′ SPAN TRUSS

	Existing Dead Load (KSI)	Proposed Dead Load + Live Load w/o Post Tensioning (KSI)	Proposed Dead Load With Post Tensioning (KSI)	Allowable Stress (KSI)
Bottom Chord	11.9	16.0	10.9	16.0
Diagonal	11.2	15.2	10.5	16.0
Top Chord	9.4	13.3	13.3	13.7
Verticals	3.2	4.9	4.9	14.6

In the accompanying table total dead and live load stresses with and without post tensioning are shown. The top chord and verticals which are in compression and are not aided by the post tensioning are obviously subjected to a higher stress under proposed loading, however as can be observed, the stresses are below the allowables.

In order to properly monitor the applied tension into the cables, permanent strain gauges are to be installed into the system. The gauges are to be routed to a permanent connection box. A portable load indicator, when connected, will display directly the load in the strands. The measurements can be carried out on a continual basis by City personnel and the monitoring system will serve as a "laboratory" to keep a constant watch on the behavior of the structure.

The cost for installing the strands on twelve (12) trusses is estimated at $350,000. A total of 15,000 LF of strands is required. Construction is to start in late 1990 and be completed by late 1991.

In summary, it is believed this system of installing strands to add to the load carrying capacity of tension members is a simple and economical method. Post tensioning allows for rehabilitation of structures not otherwise capable of being upgraded, as conventional methods are difficult and expensive to accomplish because of size, access and connection constraints at the pins.

NORFOLK SOUTHERN'S REPLACEMENT OF LAKE PONCHARTRAIN TRESTLE**

By: R. A. Tallent*

For those of you who are unfamiliar with Lake Pontchartrain, it is a large brackish lake located in southern Louisiana. New Orleans is located on the southeastern shore, and the Norfolk Southern trestle crosses the lake on the far east side near Highway 11.

Construction of the existing trestle started in February of 1882 and was completed in September 1883. The original trestle was about 21.6 miles long. The 2.3-mile long north approach, which was placed on fill was completed in 1891. The 13.5-mile long south approach was filled hydraulically by a dredge and was completed in 1896. Over the years, more and more of the railroad has been placed on fill, leaving only 5.8 miles of trestle at the start of this project. This is still believed to be the longest timber trestle in use today in North America.

Photo 1. Completed Concrete Ballast Deck Section

In 1919, a program was started to drive piling for intermediate bents in between the original bents. Over the years, this procedure has resulted in the total reconstruction of the trestle. The current trestle consists of five and six pile bents, with a mixture of timber and concrete caps. There are concrete spans located every 1,000 feet to limit the extent of fire damage.

Alfred Benesch and Company was asked to work with the Norfolk Southern in designing a replacement structure that could be built in line without interruption of rail traffic and withstand the corrosive environment of brackish water. The major railroad design requirements included the following:

1. Cooper E80 live load with impact.
2. A 16-foot wide ballasted deck to allow spotting of replacement ties alongside the operated track.
3. A minimum of nine inches of ballast under the tie.
4. Eighty-year serivce life with minimum maintenance.
5. Positive tie-down for the superstructure to resist high water caused by hurricane winds.

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*Process Engineer Track, Norfolk Southern

**Presentation to the A.R.E.A. Fall Technical Conference, Jacksonville, Florida, October 25, 1990

The major railroad construction requirements included the following:

1. Any element of construction must be completed within a four-hour work window and be able to pass trains. This includes driving piles, placing and pouring pile caps and, most critically, the removal of existing trestle and the placement of new spans.
2. Because of the continuous welded rail can't be cut, the lengths of the spans are restricted.
3. Because the work is to be performed by railroad forces, the components must be easily handled.
4. The superstructure must be able to accommodate varying span lengths to avoid the several generations of piles that have been driven over the years.

The starting point of the project was to find structural materials and components that could survive 80 years in this corrosive environment. The first step was to perform a literature search to provide state-of-the-art information in this area. We worked through an information consultant who, using computer data bases, has access to millions of abstracts. Using key words such as bridges, steel, concrete and corrosion, we searched the following data bases:

1. National Technical Information Service
2. Transportation Research in Progress
3. Engineering Index Compendex
4. Federal Research in Progress
5. Metal Abstracts
6. Surface Coating Abstracts

We received a bound set of abstracts from the information consultant, each of which had one or a combination of our key words. From the abstracts, we determined which papers would be of value and then obtained them. The following conclusions were reached:

1. There is no way to use steel in a corrosive environment without a periodic maintenance program.
2. The best paint systems last about 15 years, and that is in a non-corrosive environment.
3. Cathodic protection is not practical for this type of structure, and it should also be considered a maintenance intensive solution.
4. When a service life of 80 years in a corrosive environment with minimum maintenance is required, concrete is the clear choice.
5. Reinforced concrete has been used successfully in brackish and marine environments for years.

To properly evaluate the various methods of corrosion prevention in reinforced concrete, it is important to understand the electrochemical corrosion process. The three requirements for corrosion are moisture, oxygen, and electrochemical potential. The high alkalinity of cement paste creates a protective iron oxide film on the reinforcing steel. However, high concentrations of chloride ions from the brackish water can break down this protective film, creating a galvanic cell.

The two basic ways to prevent corrosion are to limit the penetration of chloride ions into the concrete and to protect the reinforcing steel by coating or cathodic protections. The best way to prevent chloride ion penetration is to use high quality concrete. Not only does it give you high compressive strength, but it gives you high alkalinity and low permeability. High alkalinity provides a stronger protective film that requires a higher concentration of chloride ions to break it down while lower permeability allows less chloride ions to penetrate the concrete. High quality concrete also provides good mechanical strength against abrasion, impact and spalling. To obtain a high quality concrete, a low water-to-cement ratio is required, as well as high quality, well-graded aggregates with a maximum size of three-fourths inch. By using precast concrete, we can provide the quality control required to produce a high quality concrete. Precast concrete also allows easy inspection and rejection, if required.

The second way to prevent chloride ion penetration is to protect the concrete surface. Although coatings provide an airtight, waterproof seal, they should be considered maintenance intensive. Also, if there is a break in the coating, a localized galvanic cell can be created. Silane is a product that is sprayed on, bonding with the substrate to provide a water-repellent surface. However, the service life is unknown at this time, and silane will probably have to be reapplied through a periodic maintenance program.

The second approach to preventing corrosion is to protect the reinforcing steel directly. The best and proven method is the use of an epoxy coating. Its widespread use has made it very economical. Florida Wire and Cable Company has developed a similar coating for prestressing strand. This cross-linked polymer coating has tested well in permeability, elongation and bond. The cost runs about double that of bare strand. Because it has not been in place in an actual structure for any length of time, it was decided not to use it on this project.

Now that the structural material has been investigated and chosen, we begin the second phase of the study. We needed to find a structural system that can be built within these demanding construction constraints. Repeating the more demanding constraints, the sizes and lengths of members must be easily handled so they can be placed by railroad forces using railroad equipment; all work must be done within four-hour work window, including driving piles and setting spans; and finally, all work must be done without cutting the rail.

Because of the magnitude of this project, several systems were studied, including ones that could not be handled by railroad forces. This allowed us to provide cost comparisons to performing the work with railroad forces over a 15-year construction period. The system chosen proved to be the most economical for the 15-year plan.

Photo 2. Setting Concrete Span

The superstructure chosen was two eight foot wide, prestressed concrete, double box girders sitting side by side. They are 33 inches deep and range from 24 feet to 34 feet long. They can easily be placed by railroad-owned cranes. The depth of the box was held to 33 inches to provide freeboard over high water. The top and bottom slabs are eight inches thick, while the exterior webs are seven and one-half inches thick and the interior web is 10 inches thick. The ballast stop is 12 inches high and nine inches wide. The strand pattern was chosen for ease in fabrication. Strands are spaced at multiples of two inches for every span length. This means that only one end plate is required for the entire project. For the shorter spans, where fewer strands are required, the extra holes can be plugged.

To provide an 80-year service life with minimum maintenance, the following were specified:

1. High quality concrete.
2. Four inches of cover for all prestressing strands.
3. Epoxy coating on all non-prestressed steel.
4. Design for no tension in the box girder to eliminate cracks.
5. Four thousand (4,000) psi compressive strength in the concrete at transfer and 7,000 psi compressive strength in concrete at 28 days.
6. Prestressing steel - one-half inch diameter stress-relieved strands with a minimum ultimate strength of 270,000 psi.

The substructure chosen was a six-pile bent with composite piles and cast-in-place pile cap. The composite piles can be driven within the work window and the top of the cap is below the trestle so that it can be constructed without interrupting rail traffic.

The pile cap is cast-in-place concrete construction. Four thousand (4,000) psi concrete was specified because of the lack of quality control for ready-mix or site concrete. It also provides a more workable mix in the field. All reinforcing steel is epoxy coated with a minimum of four inches of cover. The pile is seven feet six inches wide, three feet six inches deep and seventeen feet six inches long. The pile embedment ranges from a minimum of two feet zero inches to a maximum of two feet nine inches. This ensures moment capacity while hopefully giving us enough leeway to avoid cutting off the concrete piles.

Because the piles were expected to be driven between 95 and 100 feet below the top of tie, a splice was inevitable. This allowed us to use a 24-inch square, 35 foot long concrete pile in the corrosive zones and a 12-inch diameter pipe pile in the soil zone. The connection between the two dissimilar pile types was accomplished by embedding a short section of pipe in the concrete pile. To prevent galvanic cell action, this short section of pile is coated with coal tar epoxy inside and out. This section is connected to the already driven pipe pile with a slip fit splice that is welded all around with a one-quarter inch fillet. The concrete section of pile is conventionally reinforced with epoxy-coated rebar. The compressive strength of the concrete is 7,000 psi at 28 days. Reinforcing was added inside the embedded pipe section to provide additional stiffness.

Positive tie-down of the superstructure is provided by a #11 bar that is cast into the pile cap with a threaded coupler at its end. A hoop is provided in the box girder when it is cast. After the box is placed, a #11 bar is slipped in the hoop and threaded into the coupler.

It was decided early on that we would use our own construction people instead of going to outside contractors when replacing the timber structure on Lake Pontchartrain. The problem is most of our people have a lot of experience with timber structures and with steel structures, and a number of crews could build ballast deck structures but not very many of us have had much experience with concrete. Therefore, the learning curve became a little longer than we had expected. We had to acquire different types of equipment and we had to go about a different way of maintaining the existing structure. Previously, by 1986, we had completely replaced the welded rail that was on the trestle with 132 pound welded rail.

We are building the replacement concrete structure in 1,000 foot segments so we have a lot of structure to maintain while we are in the building mode of replacing the timber structure. There is a fire break every 1,000 feet on the existing trestle so in case of fire we know that we are only going to lose a 1,000 foot section of structure at one time. We are tying the concrete construction into the existing concrete fire break. We come in and put down a precast riser, block set it and use epoxy grout to fasten to the existing fire break.

When we first started planning, we decided we could not keep our maintenance people traversing the bridge every day if we were going to have this large construction project going on. So we decided to move a lot of our maintenance operation to the water. We looked around and found some sectional barges built by Shugart Manufacturing Company of Chester, South Carolina. The main reason we looked at these was the way the spuds were adapted, with the spuds being operated hydraulically. We moved our crane out onto the barge and moved from one location to another as we needed to change the caps with our capping crew.

In another section barge that also has a hydraulic spud arrangement on it we built a track and put our Kershaw bridge tie crane on it. We used it to change bridge ties. Also, when we are in the concrete construction mode, we have to take out the bridge ties and replace them with regular track ties and we do this from the water also.

We had a DE40 American locomotive crane in pile driver service. The problem was we were using the MKT-DE-30-2033 hammer which was great for driving steel, but when we got to the 24 inch concrete pile, the mass of the concrete absorbed all the energy; therefore, we couldn't move the concrete. So, in answer to that, we added a FEC3000-63000 footpound diesel hammer to our existing FEC leads and used that to drive our concrete pile. At present, we are driving 90 feet of steel before we attach the concrete pile.

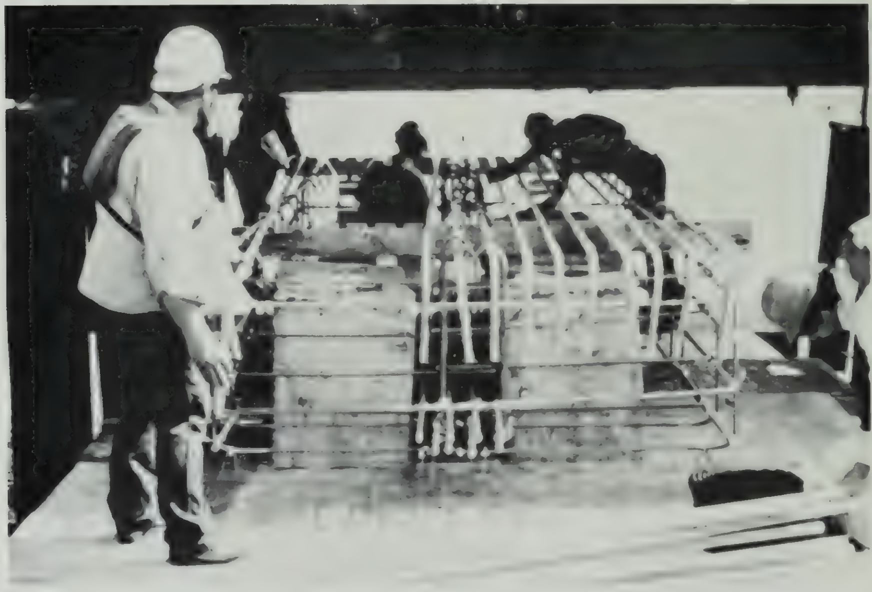

Photo 3. Tieing Rebar for the Concrete Cap

In a completed bent, with six piles to the bent, outside piles are battered transversely and longitudinally. Once the piles are completed, the capping operation starts. To add to our construction fleet we purchased two more sectionalized barges to use with our construction people. One was with a crane with our capping crew and the other barge was used with our pile driving crew. All of our piles and all of our components are brought to us by barge.

Friction collars will be placed around the concrete piles before we put our beams on to make the bottom of the cap form. Sometimes, at an unusually high tide, these collars are underwater and the bolted connection for the collar necessitates people having to get into the water. Once the beams are in place, we are ready to start the bottom portion of the cap form. We use timber bottom forms. A new crane was just purchased for our capping operation. It has a hydraulic tool circuit on it. We use a hydraulic impactor wrench to tighten the friction collars because sometimes those bolts have to be tightened under water.

Here (Photo 3) we are putting rebar into the area getting the caps ready to pour. The first cap we tied took two days, now all the steel rebars are tied in two hours.

A steel side form (Photo 4) was made by Shugart Manufacturing. They do a lot of specialized cap forming. This one was designed where we could open it from either end. They set it in place, closed the other end, bolted it in place and it was ready for the pour.

Photo 4. Setting Steel Side Form

The next big problem was how to get the concrete to the cap. We looked into building a ready-mix drum mounted on high rail units. When the price got up to $250,000 per unit, we decided there had to be a cheaper way. One other way was a TTX car. Standard Materials, the ready mix plant in Slidell, Louisiana, allowed us to load the ready-mix trucks on a TTX car. We brought the locomotive and TTX car in, parked it at the ramp and loaded the trucks.

We figured a locomotive crane would not give us the versatility we needed in setting spans. We would have to bring the 35 ton spans out by rail and this was going to cut down on our construction time. We wanted everything delivered by water. It was going to be hard to pick up a 35 ton span with a 40 ton locomotive so we needed more equipment.

What is commonly referred to as the Shugart doughnut crane (Photo 5) was being used in South Carolina in bridge construction work. It is the only one in the world that exists at this time. It is designed and built with a hollow open section for stability in rough water. It has an American 700 series crane on the back. The boom foot attachment in the front is permanently fixed in place. You maneuver the crane on a center spud, and you have two hydraulic power thrusters, one on each side. The operator's cabin is in this area where he controls all the movement and power. He can lift and lower his spuds, back the crane out, maneuver it forward, swing it 360 degrees around the center spud. This gave us over 100 tons capacity and at 123 feet from the center, we could pick up 40 tons. Eighty feet from the boom foot we could pick up 4 tons. We went one step further and equipped it with 100 feet of semi-fixed vertical travel FEC leads with another F3000 hammer. We used our locomotive crane to drive our steel pile and we used the doughnut crane to drive our concrete pile. This crane picks up, and pivots around the center spud. It has two spuds on either side. The one on the other side is called the spotting spuds. Once it is in position where it needs to be, they are dropped to stay in that position. All of this, even the movement of the batter in the leads, is controlled by one man in the cab of the doughnut crane. It can reach all the way across the trestle, drive the outside pile on the other side and it can also set the off side spans.

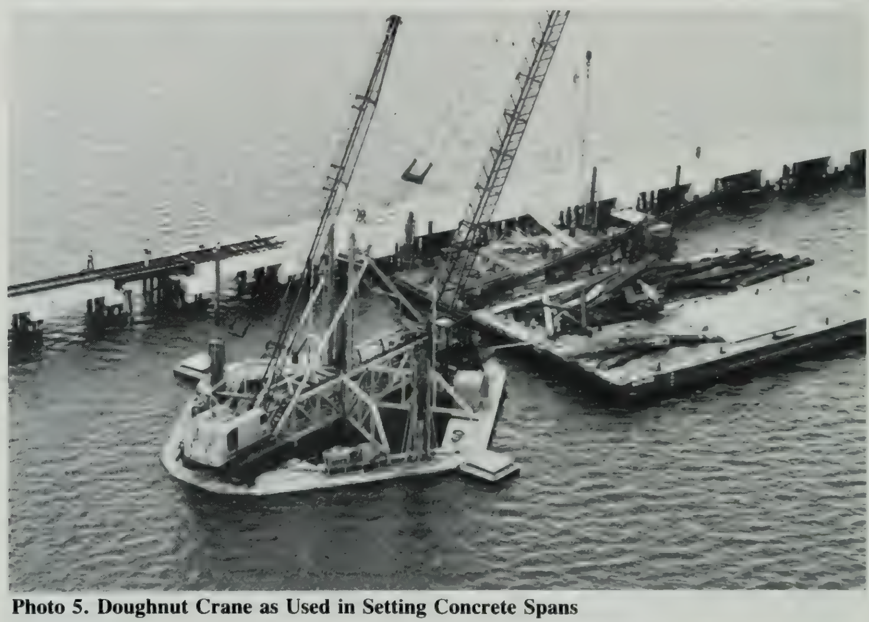

Photo 5. Doughnut Crane as Used in Setting Concrete Spans

Concerning the tearing out procedure, we removed the timber stringers, took off the timber caps, and got ready for the concrete span. We did not cut the ribbon rail. We jacked the rail up and slid the spans underneath. The concrete spans are delivered by barge. They are now being manufactured in Slidell Louisiana, by Standard Materials. Standard Materials is located on Bayou Bonafacu which empties into Lake Pontchartrain. Delivery time is about two hours.

In setting the spans the doughnut crane is very stable. However, on Lake Pontchartrain we are subject to quick, harsh storms and we have had water up to the bottom of the timber caps during hurricane season. Hurricane warnings cause us a big problem. We had two of our barges out and the hurricane went towards Texas and then turned around and came back at us. We tried to get some of our equipment off and the wind got up to 75 miles an hour, and the waves rose to six and eight feet so our people had to abandon our utility boat. The storm snapped the spuds off two of our sectionalized barges. We had $26,000 worth of damage to the barges and utility boat. When a hurricane is forecasted now, even if it just talked about, we go hide.

Our piling and all of our components, as I said before, are brought to us by barge. When we first got the doughnut crane we were told how stable it was in a marine environment. They also say you've exceeded the limits of this crane when the operator's feet are in the water. He sits 10 feet off the deck. We did put a portable hydraulic power unit on two of our sectionalized barges because we had more work than the utility boat could do in moving four sectionalized barges, some of the equipment and the components. So along with Shugart Manufacturing we developed a portable hydraulic power unit that we just set on the deck of one of the sectionalized barges, which also has hydraulic controls in the front where we can operate and raise and lower our spuds. When this unit is needed on another sectionalized barge, we just pick it up and set it on the other one. With this unit, we can move along the side of the bridge, lower the spuds and go to work.

We have a rotary dump high rail truck that brings our ballast out to us. As we set a span, we go ahead and ballast and get as much of the rock on as we can at that time. Later we come back in and surface the track.

The last thing we do when we get a segment set is come back in and snap off the old timber piles at the mud line and remove them. We are trying to work this in 1,000 foot segments.

I thank you for your attention today and the opportunity to present this project on the Norfolk Southern.

Summary of Heavy Axle Load Tests To Date At Transportation Test Center**

FAST/HAL RAIL PERFORMANCE TEST

By: J. Hannafious*

INTRODUCTION

Rail wear and rail fatigue tests have been conducted at the Transportation Test Center (TTC), Pueblo, Colorado, on the Facility for Accelerated Service Testing (FAST), High Tonnage Loop (HTL) since its installation in 1985. The first HTL experiement to investigate rail wear and fatigue was the Defect Occurrence and Growth Test (DOG).[2] The primary objective at that time was to evaluate wear and fatigue differences between various rail metallurgies, especially premium and standard rail, when exposed to dry or lubricated operating conditions. This test accumulated over 160 MGT of 33-ton axle load traffic from June 1985 until March 1988. Figure 1 shows the HTL layout.

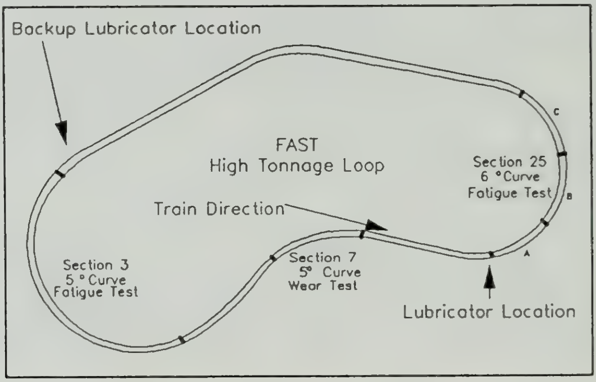

Figure 1. FAST HTL

The Heavy Axle Load Experiment (HAL) was initiated on the HTL in July 1988. The main purpose of the rail performance portion of this experiment was to investigate the effect of 39-ton axle loads (HALs) on rail fatigue and wear, especially as compared to the effects of 33-ton axle loads. It was originally determined that the HAL Test would also last 160 MGT, but has since been extended to 260 MGT. HAL testing today has accumulated tonnage of 160 MGT.

The HTL has always been operated under a lubricated regime except for special dry wear tests. When operations started in 1985, only the outside rail of the HTL was lubricated. Track grease was applied to the gage face of the high rail in Section 24 with a wayside lubricator (Figure 1). After one year of this type of lubricated operation, the FAST train derailed in Section 25. The difference in top of rail friction between the highly lubricated outside rail and the dry inside rail contributed to high truck

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*Engineer, Association of American Railroads

**At the AREA Fall Technical Conference Dr. Albert J. Reinschmidt, AAR Director of Track Research, summarized results to date of the FAST/HAL Tests at the Transportation Test Center, which were presented in their entirety at the Heavy Haul Workshop, Oct. 14-17, 1990 in Pueblo, CO. Three of the papers presented at the Workshop and summarized at the AREA Conference by Dr. Reinschmidt are published here.

**Acknowledgements are due to the AAR Research & Test Department and the International Heavy Haul Association for their sponsorship of the Workshop, and to the Federal Railroad Administration and AAR for their cosponsorship and support of the Heavy Axle Load Program at TTC.

turning forces. This combined with weak track caused fastener weakening and eventual rail roll over. To reduce the differential friction, a light oil was applied to the top of the inside rail. The standard operating procedure now is to lubricate the gage face of the high (outside) rail in Section 25 and apply light oil to the top of the low (inside) rail.

Lubrication was applied to the HTL so that fatigue tests could be conducted in Sections 3 and 25 at the same time that wear tests could be conducted in Section 7. Section 25 is a 6-degree curve with 5 inches of superelevation and Sections 3 and 7 are 5-degree curves with 4 inches of superelevation. Under the highly lubricated conditions on the outside rail, rail wear rates are extremely low which in turn makes wear evaluation difficult. However, fatigue is common where lubrication is present on high rails of curves as in Sections 25 and 3. Section 7 was originally selected for wear testing because the high rail (inside rail of the HTL) of that curve was not lubricated at all, and wear rates here were very high. After the Section 25 derailment, the light oil applied to the inside rail contaminated the high rail in Section 7. Other contamination of the inside rail comes from the train being turned every 3 MGT and lubrication on the wheels being carried over from the outside to the inside rail. Even with contamination, Section 7 high rail wear rates are high enough to make valid comparisons in relatively short time frames.

The special dry wear tests mentioned above consisted of two periods which were completed consecutively between the 33-ton axle load and 39-ton axle load tests. The 33-ton axle load dry test, which came at the end of the 33-ton axle load lubricated test, lasted for 10 MGT and was followed by 15 MGT of the 39-ton axle load dry test. At the time of operation, FAST engineers hoped that dry wear rates could be established for all curves of the HTL (Sections 3, 7 and 25).

Throughout the entire HTL operation, the train operated at 40 mph primarily in a counterclockwise direction. At this speed, the train was 2 inches over balance speed with both the 33- and 39-ton axle load test. The FAST train consisted of four or five 4-axle locomotives and 70-80 loaded, 39-ton axle load hopper cars. Part of the standard operating procedure was that the train be turned every 3 MGT. When this transpired, the train was reversed for 30 laps. This procedure was adopted for the benefit of rail fatigue tests because the train reversal places a bench mark on growing fatigue defects. These bench marks can then be used to determine fatigue growth rates. The remaining 100 MGT of operations on the HTL will be bi-directional.

DATA COLLECTION

Rail wear measurements were taken at 15 MGT intervals in Section 7 and 30 MGT intervals in Sections 3 and 25 to determine wear and metal flow characteristics. Measurements included gage wear measurements, head height wear measurements, cross-sectional rail profiles, and longitudinal rail profiles. Figure 2 illustrates the gage face wear measuring device. Accuracy of the readings is plus or minus 0.003 inches. Data is recorded at different MGT intervals and is later used to determine wear rates. Wear rates are calculated using linear regression techniques in which a computer statistically positions a linear regression line through a scattergram of data collected over several million gross ton intervals. Figure 3 is a sample scattergram with the regression line and slope of that line.

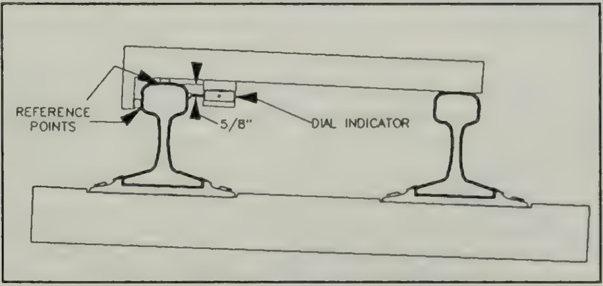

Figure 2. Gage Face Wear Measurement

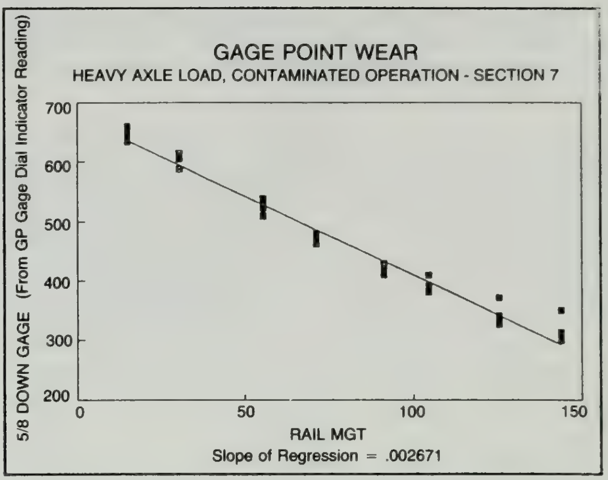

Figure 3. Scattergram With Regression

The head height loss instrument, shown in Figure 4, takes a direct measurement at the center of the rail head. This device references the base and web of the rail and also has an accuracy of plus or minus 0.003 inches. Data is collected for high and low rail of curves and processed as with the gage face wear data.

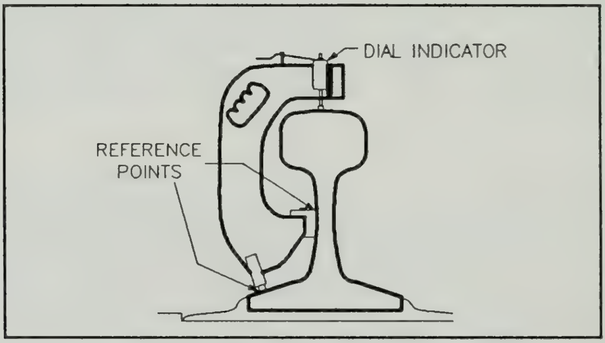

Figure 4. Head Height Loss Measurement

A device called the Yoshida Rail Profilometer traces the actual rail head profile onto a 3.5″ x 4.5″ card. These rail profiles (RP) can be overlaid to visually determine wear or metal flow characteristics.

The Longitudinal Rail Profilometer (LRP machine) records a 1 meter longitudinal tracing of a rail's running surface onto a 10-inch strip chart, i.e., the corrugation is four times the length it appears on the strip chart. Vertical fluctuations are magnified by 7.87 times so that 0.05 inch in change is reflected as 1 mm on the strip chart. The LRP machine is used to record corrugation wavelengths and depths, or welded rail end batter.

Brinell Hardness (Bhn) are taken on the running surface of rail with one of two hardness testers. Both testers are portable and provide a 3,000 kilogram (kg) load on a 10 mm ball; one is pumped manually, while the other is automatic. Measurements are taken when the rail is new, and periodically thereafter.

TTC's EM80 car measures track geometry including gage, superelevation, alignment, and profile. The EM80 gives exceptions to Federal Railroad Administration (FRA) standards, which are used to determine the need for maintenance required to keep the HTL at FRA class 4 standards.

Ultrasonic rail flaw inspections were made every 3 MGT after the train had been reversed with the wayside lubricators off. TTC's rail flaw vehicle can detect fatigue defects in the form of detail fractures as small as 3 percent of the rail head area. If a flaw is detected in test rail, it is logged into a database and inspected by ultrasonic mapping with a hand held ultrasound unit for the remainder of its life. Data is used to determine how many and what type of defects a rail had, when they occurred and how fast they grew. These efforts are particularly useful for the fatigue test in Sections 3 and 25 since Section 7 rail has yet to develop a fatigue defect in the rail itself.

In order to determine what loads a rail of a particular section of track is actually being exposed to, instrumented wheel sets are used. Data from the entire HTL is collected from these strain gaged wheels at selected times. The wheel set data includes vertical and lateral loads at any point in the track. A method used for load data collection at specific points in a track is by strain gaging the rail itself. The drawback is that only one point of a section is seen and may or may not be representative of what is happening in the rest of a test section.

RAIL WEAR

All rails in this test (RE 136, RE 132, or RE 133) were donated by numerous rail manufacturers worldwide to the FRA/AAR FAST program. Rails were installed in Section 7, tested, and removed from test when steady wear rates were established or when rails became too worn for safe train operations. Section 7 is 1,000 feet long which allowed up to eight types of rail (120 feet each) to be tested concurrently. Figure 5 illustrates which rails were previously tested and under what conditions. The position of each rail within the curve, which is shown on the left side of this figure, shows the approximate location of the rail in Section 7. Position 1 is the first rail in the curve the train passes over when traveling in a counterclockwise direction.

Several rail changes have occurred since the beginning of the HTL and are also shown in Figure 5. For example, the Alloy HH (In-Line) 360 rail was installed at 10 MGT of the 33-ton axle load test. This

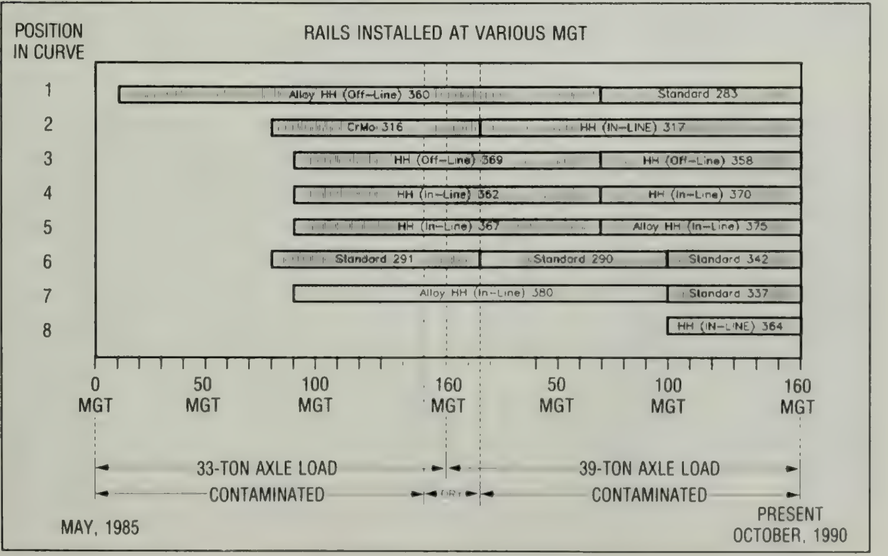

Figure 5. History of Rail in HTL Section 7

rail remained in test through the contaminated 33-ton axle load test, the dry 33-ton axle load test, the dry 39-ton axle load test, and the contaminated 39-ton axle load test. It was removed after 70 MGT of 39-ton axle load testing and replaced by the standard 283 Bhn rail. The rails currently in track were all installed during the 39-ton axle load contaminated period.

Gage Face Wear

Most railroads encounter excessive rail wear on curves with low lubrication levels, especially in the form of high rail gage face wear and low rail head crushing. Gage face wear is the predominate form of wear at FAST's Section 7 and is the focus of this report. However, the head height wear data has also been included.

Wear results from the two non-lubricated periods of the usual fully lubricated curves of Section 3 and 25 were very questionable. There are two reasons for this. First of all, in lubricated curves, the pattern of wear is different from non-lubricated curves. In fact, much of the rail which survived the first lubricated period in Sections 3 and 25 had metal flow on the gage face. Sufficient exposure to dry wear is required to produce conformal wheel/rail profiles and steady wear rates. Second, the 39-ton axle load test introduced wheels with new AAR 1:20 profiles, both 38 inch class B (Bhn 277 min to 341 max) and 38 inch class C (Bhn 321 min to 363 max). These new profiles and hardnesses were different than the 33-ton axle load test wheels (36″ class U wheel sets, Bhn ~ 249-273) with worn profiles. The two dry periods did not have sufficient tonnage in Sections 3 and 25 to produce the conformal profiles needed for wear rate comparisons. This data is excluded from wear analysis in this report.

Table 1 lists the various rails, their hardness, and gage face wear rates which were collected over the last four years in Section 7. In evaluating wear results, this report concentrates on the rails which were in track under all four operating conditions; that is, the first seven rails listed in Table 1. The standard 291 Bhn rail was replaced at the end of the dry test period with standard 290 Bhn rail. For comparison purposes, these two rails will be considered the same. Three main groups of rail types were included in the test: (1) the standard and the CrMo rails—controlled cooled rail (CC), (2) the in-line and off-line head hardened rail (HH), and (3) the in-line and off-line alloy head hardened rail (Alloy HH).

As expected, the performance of the rails within each category was similar. The CC rails had the highest wear rates followed by the HH rails and the alloy HH rails. There is one rail that took exception to the categories but in only one of the four operating periods. The alloy HH 360 rail had a much lower

Table 1. Gage Face Wear Rates

Rail		Dry Operations		Contaminated	
Metallurgy	Hardness	33-Ton Axle Load Test In/1000 MGT	39-Ton Axle Load Test In/1000 MGT	33-Ton Axle Load Test In/1000 MGT	39-Ton Axle Load Test In/1000 MGT
Alloy HH (Off-Line)	360	*0.968	2.406	1.371	2.323
CrMo	316	3.368	4.283	2.812	-
HH (Off-Line)	369	2.857	3.275	1.914	2.513
HH (In-Line)	362	2.923	2.957	1.768	2.323
HH (In-Line)	367	2.894	2.448	1.476	2.143
Standard	291	3.315	-	2.604	3.243
Alloy HH (In-Line)	380	2.493	2.519	1.067	2.081
Standard	283				2.039
HH (In-Line)	317				2.846
HH (Off-Line)	358				2.165
HH (In-Line)	370				1.826
Alloy HH (In-Line)	375				1.975
Standard	342				2.261
Standard	337				3.025
HH (In-Line)	364				2.457

*Extremely low wear rate discarded from analysis wear rate (.968 in./1,000 MGT) during the dry 33-ton axle load test than did the alloy HH 380 (2.493 in./1,000 MGT). It was expected that the alloy rails would wear moderately less than the other HH rails. But the alloy HH 360 performed so much better that it is suspected that unforeseen, and so far unknown factors had come into play which resulted in lower wear rates. This extremely low wear rate will be discarded from the analysis. However, the cause of this unusual rate is currently being investigated.

One of the secondary objectives of this experiment was to compare the wear performance of different metallurgies. This can be accomplished by examining the gage face wear figure of merits. A figure of merit (FM) is calculated by dividing the control rail wear rate by other rail wear rates. For example, during the contaminated 33-ton axle load test, the gage FM for the alloy Bhn 380 is calculated by dividing the standard 291 Bhn rate of 2.604 in./1,000 MGT by the alloy wear rate of 1.067 in./1,000 MGT. The resulting FM of 2.4 means that the alloy rail performed 140 percent better, or would last over twice as long as standard rail under the 33-ton axle load contaminated condition. If wear rates for standard rail were unavailable for use as the control rail, CrMo rail rates were substituted.

Table 2 summarizes all Section 7 gage face wear FMs (a complete FM listing is located in the Appendix; Table 1a). Under contaminated operations and 33-ton axle loads, the HH rails increased rail life from 30 percent to 80 percent over standard rail, while the alloy HH rails increased life 90 percent to 140 percent. Under contaminated 39-ton axle loads (HALs), these same rails increased life 30 percent to 50 percent and 40 percent to 60 percent, respectively. This data suggests that under very light lubrication, increasing axle loads reduces the benefit (in terms of increased rail life) of HH and alloy HH rail over standard rail.

Table 2. Gage Face Wear Figure of Merits

Rail Metallurgy	Dry		Contaminated	
	33-Ton Axle Load Test	*39-Ton Axle Load Test	33-Ton Axle Load Test	39-Ton Axle Load Test
CC	1.0	1.0	1.0	1.0
HH	1.1 to 1.2	1.3 to 1.7	1.3 to 1.8	1.3 to 1.5
Alloy HH	1.3	1.7 to 1.8	1.9 to 2.4	1.4 to 1.6

*CrMo was used as control rail.

Dry operations generated different results. During the 33-ton axle load test, HH rails increased rail life only 10 percent to 20 percent, and alloy HH rail increased rail life by 30 percent. Under 39-ton axle loads, HH rail increased life 30 percent to 70 percent, while alloy HH increased life 70 percent to 80 percent. Therefore, the benefit of using HH and alloy HH rail over standard rail under totally dry conditions increased with heavier axle loads.

Review of the FMs is appropriate for relative rail performance, but it does not display the actual effect of increasing axle loads on individual rails. Table 3 summarizes the penalty factors encountered from increasing axle loads from 33 tons to 39 tons under both dry and contaminated conditions. Penalty factors (PF) are calculated by dividing the 39-ton axle load wear rate by the 33-ton axle load wear rate (a complete PF table is in the Appendix; Table 1).

Under dry conditions, increasing the axle load increased the gage face wear rate of CC rails by 20 percent, of HH rails by 15 percent to 10 percent, and of alloy HH by zero percent. Therefore, increasing the axle load from 33 tons to 39 tons (approximately 18%) increased the gage face wear rate of standard rail proportionally, but had not apparent effect on HH and alloy HH rails.

A possible explanation for this can be offered. Under totally dry conditions and 33-ton axle loads, wear rates were extremely high no matter what type of rail was being used. This can be seen in the Table 2 in which all the FMs are near 1. When the axle load was increased, the wear of the HH and alloy HH rails remained approximately the same (see Table 3). This suggests that the wear was a surface condition and not caused from subsurface metal flow (crushing), *i.e.*, the wear overcame any crushing of the gage. However, the softer standard rail, which is more susceptible to crushing, encountered both crushing and surface wear. Thus, the wear rate of the CC increased, but not those of the premium rails.

Table 3. Gage Wear Penalty Factors for Increasing Axle Loads

Rail Metallurgy	Dry Operation	Con-taminated Operation
CC	1.2	1.2
HH	.85 to 1.1	1.3 to 1.5
Alloy HH	1.0	1.7 to 2.0

Under contaminated operations, the CC rail wear again increased proportionally with wheel load at 20 percent. However,the HH and alloy HH rail, which showed no increase of wear with increased load under dry conditions, did show an increase with contamination. The HH rail wear rate increased 30 percent to 50 percent and the alloy HH rate increased 70 percent to 100 percent. Thus, under lightly lubricated conditions, increasing the axle loads increased the wear of premium rails more than standard rails. With the contaminated condition, surface wear is not high enough to overcome crushing. Therefore, increasing the load would increase the wear rate.

Head Height Loss

The gage face wear data gives an incomplete picture of the wear that took place in Section 7. Comparisons of high and low rail head height loss PFs for increasing axle loads may help give a clearer picture. Table 2a in the Appendix lists all the head height loss rates encountered in Section 7 that can be used for these comparisons.

Table 4 summarizes the head height loss PF ratios of 39-ton axle load wear rate to 33-ton axle load wear rate (a complete PF listing is located in the Appendix; Table 2b). Under dry conditions on the high rail, increasing the axle load resulted in an increase in head height loss in all cases. The CC rate increased 30 percent, the HH increased 80 percent to 220 percent, while the alloy rate increased 70 percent to 230 percent. However, the low rail head height loss decreased in most cases for CC, HH, and Alloy HH rail. As discussed earlier, gage face wear remained approximately the same under dry conditions and increased axle loads. This indicates that high rail head height loss may become a critical factor of rail life under HALs. Also, the suggestion that surface wear overtakes any crushing under dry conditions is not valid for head height loss as it is for gage face wear.

Table 4. Head Height Loss Penalty Factors for Increasing Axle Loads

Rail Metallurgy	Dry Operation		Contaminated Operation	
	High Rail	Low Rail	High Rail	Low Rail
CC	1.3	0.6	1.4	0.9
HH	1.8 to 3.2	0.4 to 1.1	1.8 to 1.9	1.0 to 1.4
Alloy HH	1.7 to 3.3	0.3 to 1.1	1.8	0.7 to 0.9

High rail head height loss PFs under contaminated operations are very similar to the dry period PFs. The head height loss increased in all cases, again especially for HH and alloy HH rail. These PFs are also comparable to the gage wear PFs in Table 3. Increasing the axle loads under contaminated conditions increased the gage face wear and head height loss nearly the same. However, the PFs for the low rail, which are all near 1.0, suggest head height loss remained about the same.

CORRUGATIONS

Throughout the history of the HTL, Section 7 has been susceptible to corrugations on the high rail of the curve. During operation of the 33-ton axle load test, these corrugations were observed to have begun at a joint or a battered weld in standard rail and then carry through the remainder of the rail as tonnage was accumulated. There were few measurements taken during the corrugation development, but the longer the rail was in track, the deeper the corrugations became until the rail eventually had to be ground. Corrugations were also noticed in HH rails, but they were light and did not become worse with tonnage. A strip chart recording from the LRP machine is shown in Figure 6. This LRP was taken on standard rail after 71 MGT of contaminated 33-ton axle operation and was the only corrugation data taken during the period. The test site chosen had the deepest corrugations. Assuming the corrugations began to develop immediately with train operations, and a corrugation depth of .09 inches after 71 MGT, the growth rate is .0013 in/MGT.

Under contaminated 39-ton axle loads, corrugations in rail (with the same hardness of the previously discussed rail and in nearly the same location) were first noticed at rail joints after 5 MGT, and within the rail itself at 10 MGT. That is to say the corrugations began as soon as the rail was installed, but became noticeable as tonnage accumulated. Figure 7 displays the LRPs taken at the worst corrugations in this rail and at two intervals: 31 and 49 MGT. A later LRP could not be taken as the rail was ground at 49 MGT. The rate of corrugation growth shown on this chart increased from .0013 in/MGT at 0 to 31 MGT to .0017 in/MGT at 31 MGT to 49 MGT, and averaged at .0014 in/MGT. This indicates that the growth rate increases as the corrugation depth increases.

In the lubricated sections of the HTL, 33-ton axle load corrugation records were not collected. However, it is known that rails with Bhn of 280 + were in track in Sections 25 and 3 for up to 150 MGT with no heavy corrugations. Also, rail with Bhn of only 269 became extremely corrugated by 60 MGT in these same curves. Under 39-ton axle loads, corrugations in Section 25 have been confined to joint locations in standard rail and have not become severe. In Section 3, however, standard rail (similar to that in Section 7) became corrugated under 39-ton axle loads as in Section 7.

The length of corrugations can change as shown in Figure 7. The 31 MGT corrugation, 13 inches long, grew to 14 inches by 49 MGT. The two strip charts were overlaid with no actual reference point, *i.e.*, one point was matched over the other for a reference. Therefore, the acutal movement of the corrugation peaks in relation to the rail and to each other has not been determined.

To determine the rail load environment in the corrugated sections, vertical load data was collected from corrugation peaks and valleys in Section 7. Strain gages were applied to the rail at two peaks and two valleys with depths of .070 inches to .080 inches. A special consist of cars from both the 33- and 39-ton axle load tests then passed over the gages at 40 mph. The vertical load data gathered is displayed in the exceedance plot of Figure 8.

The most striking result of the corrugation load data is the difference between the peak and the valley loads. Peak and valley loads for the 33 kip wheel ranged from 11 kips to 35 kips and from 32 kips to 80 kips, respectively. Wheel loads for the peak and valley under 39 kip wheels ranged from 19 kips to 43 kips and from 65 kips to 93 kips, respectively. The increase of 6 kips in static wheel load resulted in an increased maximum load at 40 mph for the peaks of 8 kips and at the valleys of 13 kips.

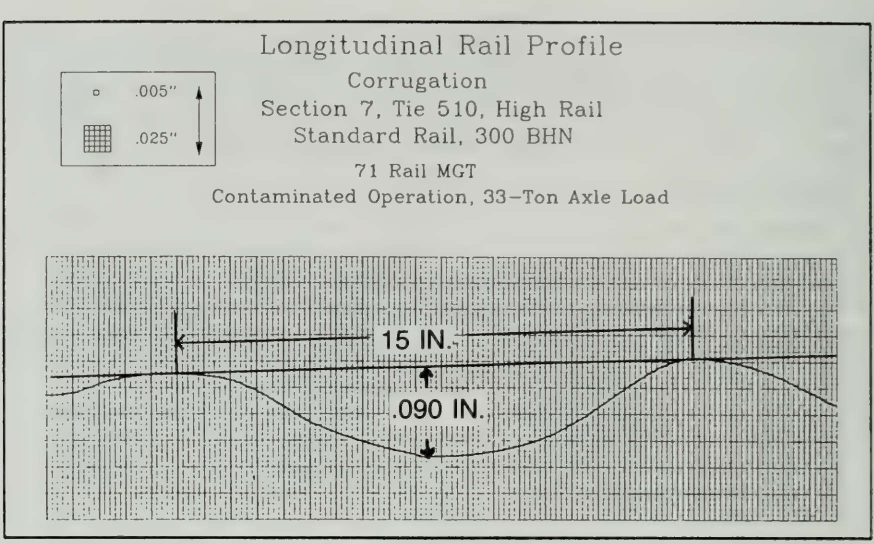

Figure 6. Corrugation Strip Chart 33-Ton Axle Load

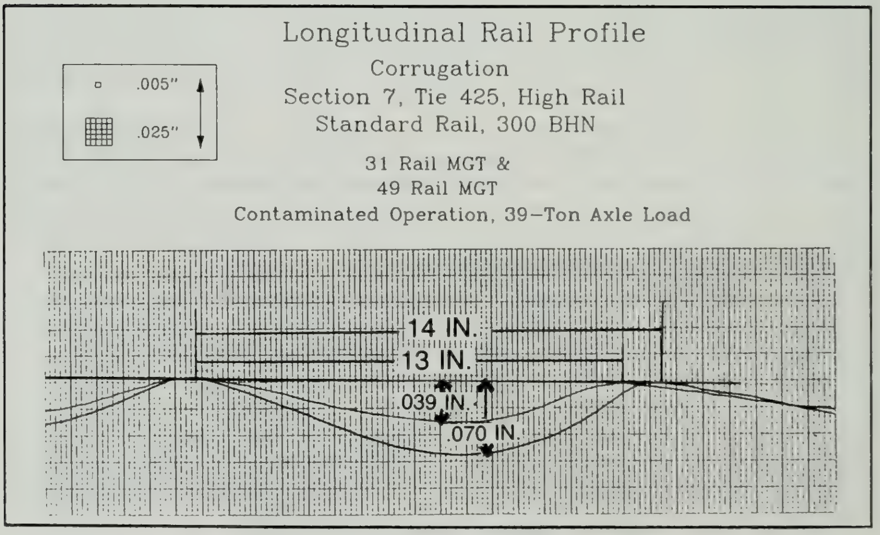

Figure 7. Corrugation Strip Chart 39-Ton Axle Load

FATIGUE

The fatigue tests on the HTL, as discussed earlier, took place mainly in Section 25. The procedure for rail fatigue testing was to place rail in track under lubricated conditions and monitor fatigue defects in the high rail of the curve. The rail was not ground unless it was part of a special grinding fatigue test. Lubricators were placed in Section 24 in order for lubrication levels to be higher at the beginning of

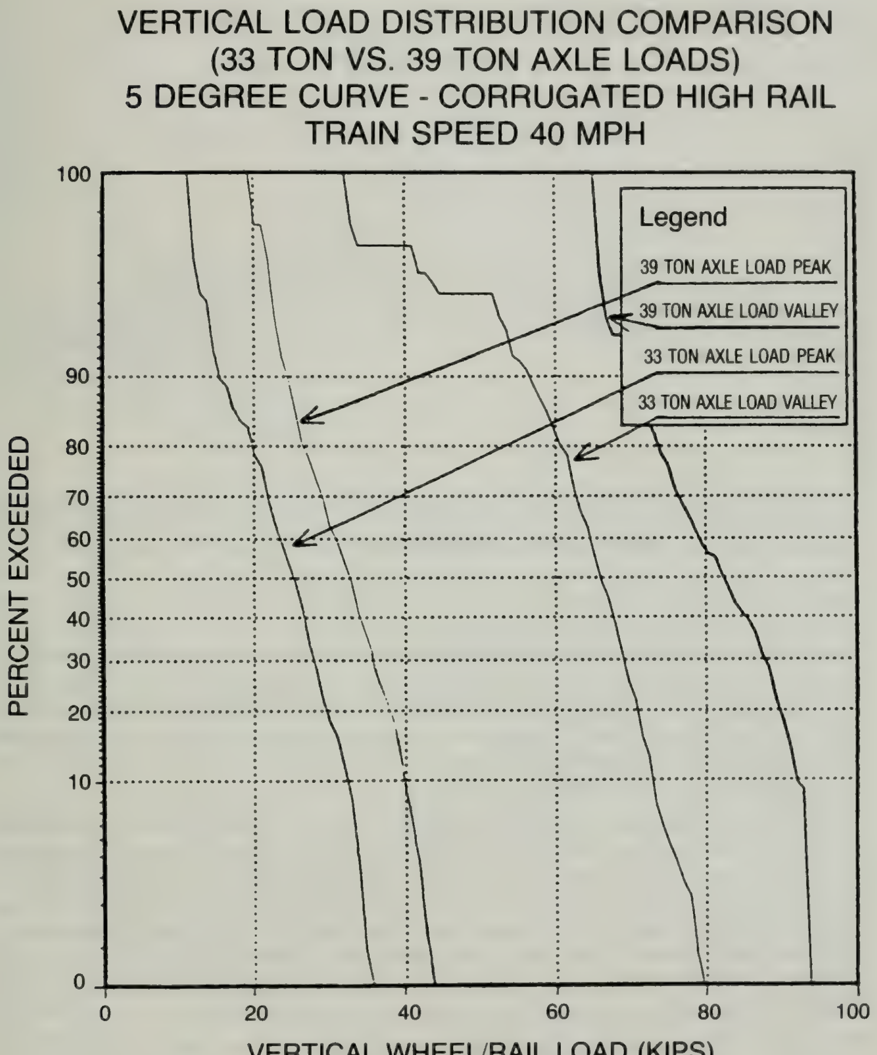

Figure 8. Vertical Load Data

Section 25 than at the end of the curve. Higher lubrication was expected to increase the initiation of rail fatigue defects. Section 25 was divided into three segments during the 33- and 39-ton axle load tests, A, B, and C, as shown in the HAL test layout in Figure 9. Segments A and C were fatigue tests in which rails were duplicated to determine the effect of lubrication level. Segment B was a grinding fatigue test. Rails have always been duplicated in Segments A and C in case there was an effect of lubrication level.

Rails for the fatigue tests were donated by various manufacturers. The standard rails had a hardness specification of 269 Bhn minimum, while the hardness of the head hardened rails was not specified. In

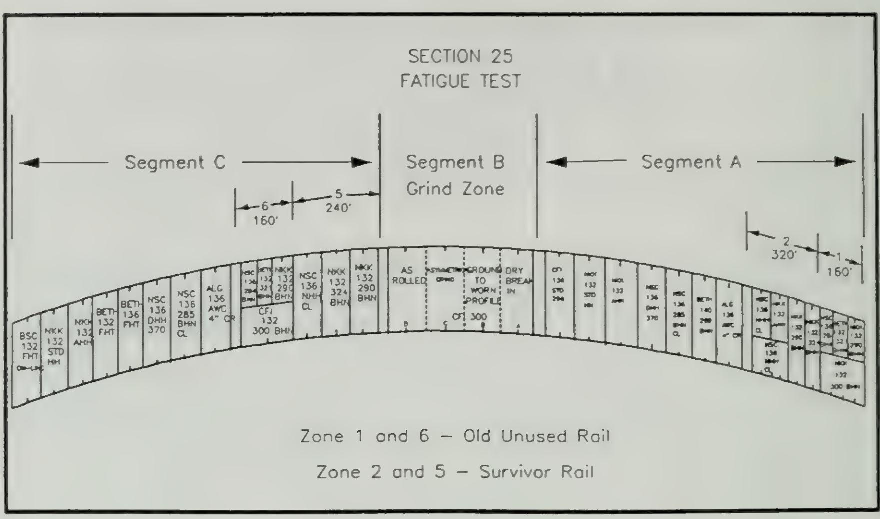

Figure 9. Section 25, Old Unused and Survivor Rail

preparation for the 33-ton axle load fatigue test, four rails of each metallurgy and manufacturer were placed in the high rail of Section 25, two rails in Segment A, and two rails in Segment C. Fatigue defects in the form of shells and detail fractures developed in many of the standard rails but very seldom in the head hardened rails. In spite of the expected effects of lubrication, the results suggested there was little or no difference in the number of defects developed in Segments A and C; therefore, this report does not distinguish between these two segments.

As stated, the main objective of the HAL Rail Performance Fatigue Test was to compare the effects of 39-ton to 33-ton axle loads. In order to do this, the same type of rail had to be installed in each test. This was done by setting aside some of the rail originally donated to the DOG Test and later installing it in the HAL Test. There was a very limited number of these "old unused" rails since the original number donated was small. Along with the old unused rails, some of the rails which had not produced defects during the 33-ton axle load test were left in track under 39-ton axle loads. These "survivor rails" were left in to increase the number of comparable rails as well as to see the effect of 39-ton axle loads on 33-ton conditioned rail.

Table 5 lists the old unused rails and the survivor rails, their average hardnesses as tested, and the total number of each that were in track during the 33- and 39-ton axle load tests. For example, there were four Bethlehem 321 Bhn rails in the high rail of Section 25 during the 33-ton axle load test, but only two during the 39-ton axle load test. All the survivor rails were reinstalled for the 39-ton axle load test and therefore had the same number in both tests. The population of these rails is small which must be kept in mind in drawing conclusions based on fatigue results.

When defects in test rail became severe enough to remove from track, some of the test rail was removed with it. Some test rail was also removed when rail end welds failed. This caused a decrease in the sample size of the rail as the test progressed. During the 39-ton axle load test, field weld failures became more common than in the 33-ton axle load test and in turn adversely affected the sample size. Also, the 39-ton axle load test sample was smaller to begin with.

Table 5. Number of Old Unused and Survivor Rails

Rail Metallurgy	Number of Rails in Track During 33- and 39-Ton Axle Load Tests	
	33-Ton Axle Load Test	39-Ton Axle Load Test
Old Unused:		
NSC Standard 284	4	4
BETH Standard 321	4	2
NKK Standard 290	4	2
Survivor:		
NSC HH 371	4	4
NKK Alloy HH 390	4	4
NKK Standard 324	4	4
NKK Standard 290	2	2

Table 6 lists the Fatigue Test results from the old unused rails and shows rail type along with the average accumulated number of detail fractures (DF) and shells per rail at 150 MGT of 33-ton and 145 MGT of 39-ton axle loads. The NSC 284 rail had three DFs and 0.5 shells per rail at 150 MGT of 33-ton axles. Under 39-ton axle loads, it had three DF and five shells at 145 MGT. The 39-ton axle loads seemed to have a very detrimental effect on this rail, especially in the form of shells. 39-ton axle loads were also very detrimental to the NKK 290. This rail developed no defects at all during the DOG test, but averaged 1.5 DFs per rail under 39-ton axle loads. Increased axle loads did not seem to be detrimental to the Bethlehem rail, possibly because of its higher hardness. This rail had three DFs and 2.75 shells per rail during the 33-ton axle load test, but only 1.5 DFs and two shells per rail during the 39-ton axle load test. This data suggests that rails which developed fatigue defect under 33-ton axle loads will do so at about the same rate or under a noticeably higher rate under 39-ton axle loads.

Another observation was made on some old, unused rail which is not listed in Table 6. These rails, which were manufactured to the old AREA hardness specification for standard rail (248 Bhn), were in all segments of Section 25 during the 33-ton axle load test and were in the beginning of Section 3 during the 39-ton axle load test. Section 3 (5 degree with moderate lubrication) is a less severe curve than Section 25 (6 degree with heavy lubrication). Rails placed in Section 3 are expected to last longer than similar rails in Section 25. The old standard rail had to be removed from the 33-ton axle load test anywhere from 60 MGT to 132 MGT. Under 39-ton axle loads, this same rail had to be removed from Section 3 at 37 MGT. Being placed at the beginning of Section 3 instead of further into the curve may have contributed to the degradation of this rail, but it seems that the increased axle load probably had the bigger effect.

Table 6. Rail Fatigue Performance Under 33- and 39-Ton Axle Loads

Rail Metallurgy	33-Ton Axle Load Test 150 MGT		39-Ton Axle Load Test 145 MGT	
	DFs/ Rail	Shells/ Rail	DFs/ Rail	Shells/ Rail
NSC Standard 284	3.0	0.5	3.0	5.0
BETH Standard 321	3.0	2.75	1.5	2.0
NKK Standard 290	0	0	1.5	0

Table 7 lists the defects that occurred in the rail that survived the 33-ton axle load test and was installed in the HAL Test. Two of the surviving metallurgies were the standard NKK rails. The NKK 324 was actually a low alloy rail while the NKK 290 (also an old unused rail) was actually an extra clean standard rail, *i.e.*, it was cleaner than most rails. Since these rails produced no defects during the 33-ton axle load test, they were installed as survivors in the HAL. The other two survivors were head hardened rails which had been in track since the start of the HTL. The standard rails were in track for a total of only 295 MGT because they were taken out during the two dry run periods. For example, in the left column of Table 7, all the rail had accumulated 150 MGT of lubricated 33-ton axle loads. In the right column, the standard rails had 295 MGT (150 MGT 33-ton and 145 MGT 39-ton axle loads) while the head hardened rails had 320 MGT (150 MGT, 33-ton axles, lubricated; 10 MGT, 33-ton axles, dry; 15 MGT, 39-ton axles, dry; and 145 MGT, 39-ton axles, lubricated).

The NSC HH 371 developed a shell in the first 150 MGT of the 33-ton axle load test. This was unusual and was the only defect any of the survivor rails developed during this test. As this rail continued in test under 39-ton axle loads, it developed a DF. As the NKK 324 continued under 39-ton axle lolads, it developed a 0.5 shell and a 0.25 DF per rail. The NKK 290 performed about the same as the old unused NKK 290; developing 1.25 DFs but no shells. The only rail to survive without any defects was the NKK alloy HH.

It is difficult to determine from these results what the effect of 39-ton axle loads was on the survivor rail. The defects that occurred under heavy axle loads could be a result of total accumulated tonnage or of the 39-ton axle loads. It can be said, however, that rails which seldom developed fatigue defects under 33-ton axle loads began to fatigue when later exposed to 39-ton axle loads.

Table 7. Fatigue Defects Survivor Rail 150 MGT of 33-Ton Loads and Subsequent 145 MGT 39-Ton Loads

Rail Metallurgy	33-Ton Axle Load Test 150 MGT		39-Ton Axle Load Test Additional 145 MGT (Add 25 MGT more for HH Rails)	
	DFs/ Rail	Shells/ Rail	DFs/ Rail	Shells/ Rail
NSC HH 371	0	0.25	0.25	0.25
NKK Alloy HH 390	0	0	0	0
NKK Standard 324	0	0	0.25	0.5
NKK Standard 290	0	0	1.25	0

CONCLUSIONS

Several important wear comparisons between 33-and 39-ton axle loads have come from the HTL tests. These observations have implications for revenue service.

Wear

The evaluation of Section 7 wear results suggests that increasing axle loads from 33 tons to 39 tons under contaminated conditions:

1. Reduced the rail life benefit (in terms of gage face wear) offered by premium rails over standard rails.

2. Increased the rate of high rail head height loss and had little effect on low rail head height loss.
3. Relatively increased the rate of high rail head height loss nearly the same as the rate of gage face wear.

Increasing axle loads under totally dry conditions:

1. Increased the rail life benefit (in terms of gage face wear) offered by premium rails over standard rails.
2. Increased the rate of high rail head height loss and reduced low rail head height loss rates.
3. Increased the rate of high rail head height loss relatively more than the rate of gage face wear.

Corrugations

The 6 kip increase in static wheel load caused vertical loads in the bottom of .070 inches to .080 inches corrugations to increase from maximums of 80 kips under 33 kip wheels to 93 kips under 39 kip wheels. This may have implications for other areas of the track structure as well, such as faster tie/fastener and ballast degradation.

Fatigue

The Fatigue Test of Section 25 has pointed out some aspects of increasing axle load.

1. The low alloy and extra clean standard rails that survived the 33-ton axle load test with no defects began to develop shells and detail fractures when subsequently exposed to HALs.
2. Intermediate strength standard rails, which developed fatigue defects under 33-ton axle loads, performed noticeably worse under HALs in one case, and performed about the same under HALs in another case.
3. One old unused extra clean standard rail, which did not develop defects by 160 MGT of 33-ton axle loads, developed numerous detail fractures by 145 MGT of HALs.

REFERENCES

1. Reiff, R. P. "Heavy Haul Workshop FAST/HAL Open House," *Pre-Conference Proceedings*. Workshop on Heavy Axle Loads at the Sangre de Cristo Arts and Conference Center and the Transportation Test Center, Pueblo, Colorado, October 14-17, 1990.
2. Clayton, P. and G. Brave. "Fatigue Defect Origination and Growth Experiment at the Facility for Accelerated Service Testing," 1985-87, unpublished.

APPENDIX

Table 1a. Section 7 Gage Face Wear Figure of Merit

Control Rail Wear Rate/Wear Rate

Rail Metallurgy	Original Brinell Hardness	33-Ton Axle Load Test Dry Operation	39-Ton Axle Load Test Dry Operation	33-Ton Axle Load Test Contaminated Operation	39-Ton Axle Load Test Contaminated Operation
Alloy HH (Off-Line)	360	-	1.8	1.9	1.4
CrMo	316	1.0	1.0	0.9	-
HH (Off-Line)	369	1.2	1.3	1.3	1.3
HH (In-Line)	362	1.1	1.4	1.5	1.4
HH (In-Line)	367	1.1	1.7	1.8	1.5
Standard (Control)	291	1.0	-	1.0	1.0
Alloy HH (In-Line)	380	1.3	1.7	2.4	1.6
Standard	283				1.6
HH (In-Line)	317				1.1
HH (Off-Line)	358				1.5
HH (In-Line)	370				1.8
Alloy HH (In-Line)	375				1.6
Standard	342				1.4
Standard	337				1.1
HH (In-Line)	364				1.3

Table 1b. Section 7 Gage Face Wear Penalty Factors

39-Ton Axle Load Wear Rate/33-Ton Axle Load Wear Rate

Rail Metallurgy	Brinell Hardness	Dry Operation	Contaminated Operation
Alloy HH (Off-Line)	360	-	1.694
CrMo	316	1.272	-
HH (Off-Line)	369	1.146	1.295
HH (In-Line)	362	1.012	1.301
HH (In-Line)	367	0.846	1.452
Standard (Control)	291	-	1.245
Alloy HH (In-Line)	380	1.010	1.950
Standard	283		
HH (In-Line)	317		
HH (Off-Line)	358		
HH (In-Line)	370		
Alloy HH (In-Line)	375		
Standard	342		
Standard	337		
HH (In-Line)	364		

Table 2a. Section 7 Head Height Loss Wear Rates

Rail Metallurgy	Original Brinell Hardness	33-Ton Axle Load Test Dry Operation (IN./1000 MGT)		39-Ton Axle Load Test Dry Operation (IN./1000 MGT)		33-Ton Axle Load Test Contaminated Operation (IN./1000 MGT)		39-Ton Axle Load Test Contaminated Operation (IN./1000 MGT)	
		High Rail	Low Rail	High Rail	Low Rail	High Rail	Low Rail	High Rail	Low Rail
Alloy HH (Off-Line)	360	1.101	0.514	1.857	0.546	0.242	0.0850	0.440	.0780
CrMo	316	1.740	2.186	2.234	1.250	0.741	0.282	-	-
HH (Off-Line)	369	0.488	0.755	1.128	0.334	0.263	0.201	0.506	0.221
HH (In-Line)	362	0.727	0.941	1.299	0.540	0.293	0.165	0.516	0.167
HH (In-Line)	367	0.474	0.401	1.533	0.460	0.273	0.172	0.517	0.234
Standard (Control)	291	1.701	1.900	-	-	1.039	0.472	1.462	0.436
Alloy HH (In-Line)	380	0.430	0.479	1.437	0.162	0.251	0.192	0.463	0.143
Standard	283							0.874	0.369
HH (In-Line)	317							0.522	0.177
HH (Off-Line)	358							0.285	0.127
HH (In-Line)	370							0.116	0.035
Alloy HH (In-Line)	375							0.105	0.031
Standard	342							0.751	0.367
Standard	337							1.414	0.570
HH (In-Line)	364							0.374	0.216

Table 2b. Section 7 Head Height Loss Penalty Factors for Increasing Axle Loads

39-Ton Axle Load Wear Rate/33-Ton Axle Load Wear Rate

Rail Metallurgy	Bhn	Dry Operation		Contaminated Operation	
		High Rail	Low Rail	High Rail	Low Rail
Alloy HH (Off-Line)	360	1.7	1.1	1.8	0.9
CrMo	316	1.3	0.6	-	-
HH (Off-Line)	369	2.3	0.4	1.9	1.1
HH (In-Line)	362	1.8	0.6	1.8	1.0
HH (In-Line)	367	3.2	1.1	1.9	1.4
Standard	291	-	-	1.4	0.9
Alloy HH (In-Line)	380	3.3	0.3	1.8	0.7

FAST/HAL TURNOUT PERFORMANCE EXPERIMENT

By: D. M. Read*

INTRODUCTION

This paper describes results of the Turnout and Frog Performance Experiment which is currently underway as part of the Heavy Axle Load (HAL) Program at the Facility for Accelerated Service Testing (FAST), Transportation Test Center (TTC), Pueblo, Colorado.[1] The experiment objective is to document the performance of turnouts on the FAST High Tonnage Loop (HTL) under 39-ton axle loads. Of principal interest is the service life and maintenance requirements of turnout components, including components of standard and premium materials.

Determination of turnout performance was based on quantitative and empirical data, including:

— Dynamic wheel/rail forces in the turnouts as measured with instrumented wheel sets
— Visual inspection of component condition
— Maintenance demand

DESCRIPTION OF THE TEST ENVIRONMENT AND COMPONENTS

The HTL contains four turnouts — two No. 10 turnouts, which connect the HTL with FAST Facility Tracks and the Railroad Test Track (RTT), and two No. 20 turnouts, which are located at each end of a siding known as the Bypass Track. The two No. 20 turnouts are considered to be the primary test turnouts, although component failures and maintenance are documented on all four turnouts. The location of each turnout on the HTL is shown in Figure 1. As indicated in the figure, each turnout was subject to either facing or trailing point traffic due to the single direction operation of the HAL train.

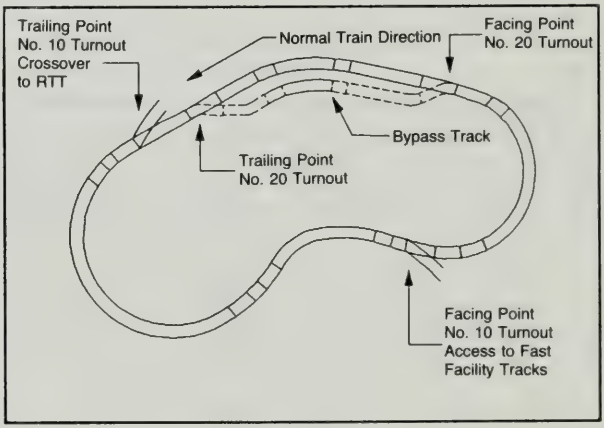

Figure 1. Location of Turnouts on the High Tonnage Loop

The facing point No. 20 turnout was installed new at the beginning of HAL testing, while the trailing point No. 20 had been in the HTL for 160 MGT of 33-ton axle load service and was relocated at the start of the program. The No. 10 turnouts were installed early in the FAST program during the 33-ton axle load phase and were rebuilt several times over the course of 33-ton axle load testing. Neither of the No. 10 turnouts were upgraded at the start of HAL testing.

At 100 MGT, the No. 20 facing point turnout, which was constructed of standard material components, was removed and replaced with a turnout of premium components. This turnout was pre-assembled by Bethlehem Steel Corp. at the Steelton plant and shipped to TTC in five panels. The panels were installed and the rail at each panel junction was thermite welded to form a completely welded rail turnout. The premium component turnout has accumulated 60 MGT of HAL traffic. The standard component No. 20 turnout at the other end of the Bypass Track remained in service throughout the 160 MGT test period with failed components replaced as needed.

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*Senior Engineer, Association of American Railroads

Of the 160 MGT of HAL traffic to pass over the turnouts, 51 MGT, or 32 percent, has been on the diverging side of the No. 20 turnouts. The train operates through the diverging side of the No. 20 turnouts at the normal FAST operating speed of 40 mph. A description of each turnout in track during the HAL program is included in Table 1.

Table 1. Description of HTL Turnouts

No.	Facing/ Trailing Point	Frog Type	Switch	Fasteners	HAL Tonnage (MGT)
20	Facing	RBM thin wall w/300 HB wing and heel rails hook twin tie plates 15' Bolted Tee Rail Guard Rails	39' curved Sampson w/1.25" reinforcing 300 HB rail graduated risers	Cut spike	100*
20	Facing	RBM explosion hardened w/fully heat treated wing and heel rails frog and gage plates	50'11" thick-web fully heat treated rail uniform risers	Pandrol E clip	60
20	Trailing	RBM heavy wall w/300 HB wing and heel rails hook twin tie plates 23' Hook Flange Guard Rails	39' curved Sampson w/1.25" reinforcing 300 HB rail graduated risers	Cut spike	160**
10	Facing	RBM heavy wall w/300 HB wing and heel rails hook twin tie plates	16'6" straight w/1.25" reinforcing 300 HB rail graduated risers	Cut spike	160**
10	Trailing	RBM heavy wall w/300 HB wing and heel rails hook twin tie plates	16'6" straight w/1.25" reinforcing 300 HB rail graduated risers	Cut spike	160**

* Turnout removed at 100 MGT to allow installation of premium component turnout

** Turnout received 160 MGT of 33-ton axle load traffic prior to HAL program

TURNOUT WHEEL/RAIL FORCES

Continuous vertical and lateral wheel force data was collected with a pair of 38-inch instrumented wheel sets installed in the leading truck of a 39-ton axle load car from the HAL consist. Time history plots of the lead axle lateral, vertical and, L/V ratio recorded throughout the facing point No. 20 turnout are shown in Figures 2, 3, and 4 respectively. The data was collected at the premium component turnout with the instrumented wheel set car proceeding through the diverging side at 40 mph. The lateral force spike at the point of switch is caused by the switch entry angle and is typical of AREA geometry turnouts.[2] The negative lateral forces produced by the switch-point side wheel ahead of the frog coincides with the frog tangent and the guard rail. The cyclic vertical forces shown in Figure 3 give an indication of car dynamics within the turnout.

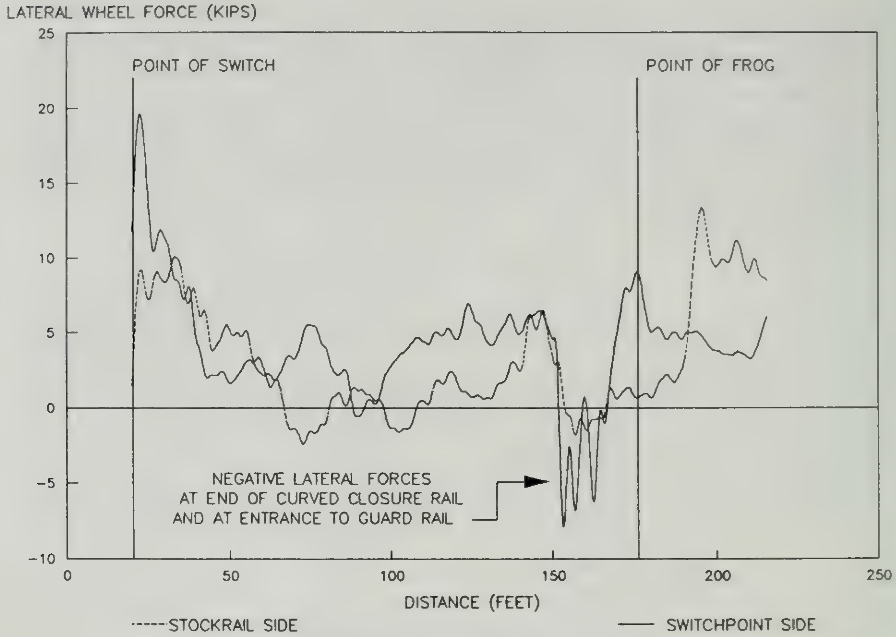

Figure 2. Lead Axle Lateral Wheel Force Data Through No. 20 Turnout Facing Point Diverging Side Move at 40 mph

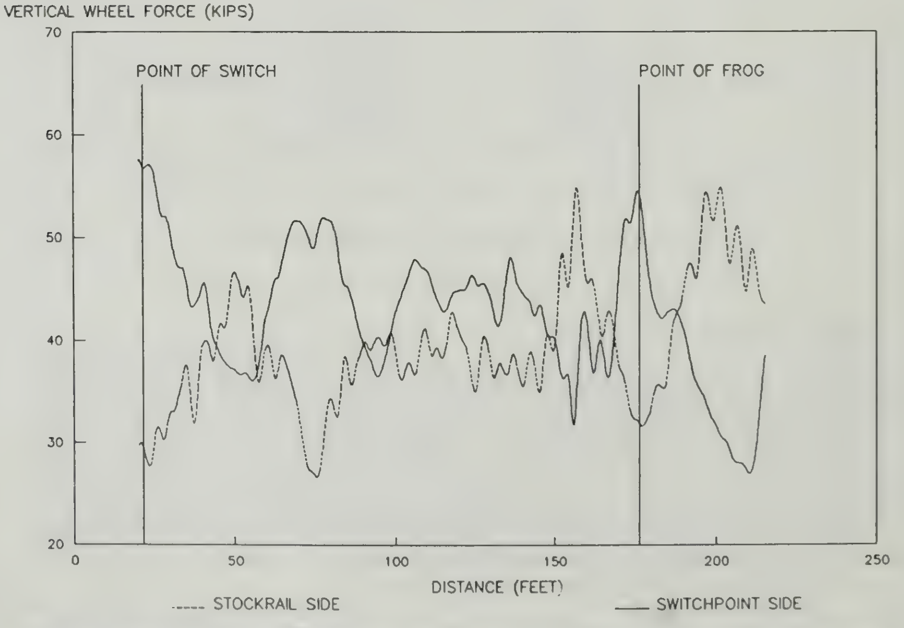

Figure 3. Lead Axle Vertical Wheel Force Data Through No. 20 Turnout Facing Point Diverging Side Move at 40 mph

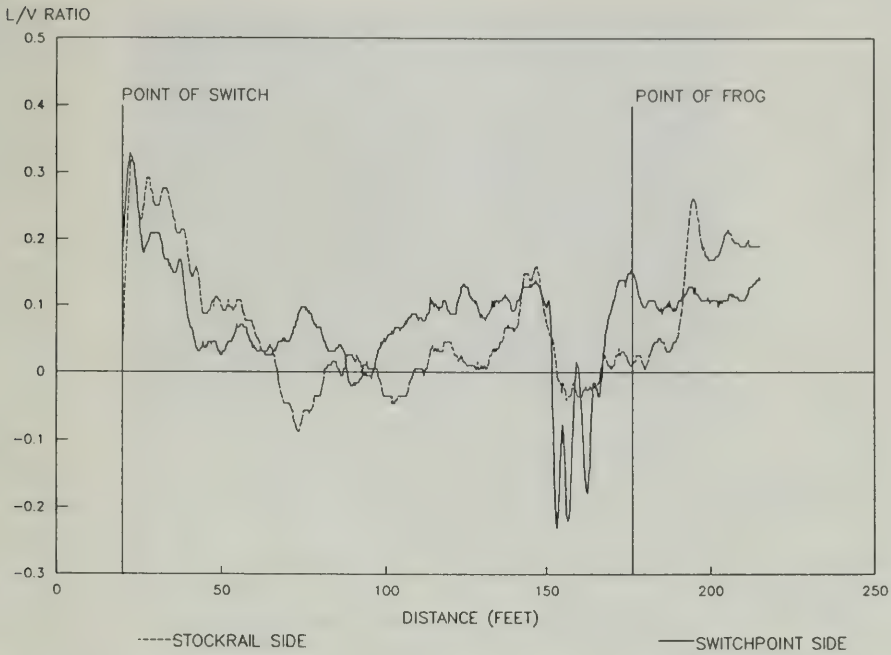

Figure 4. Lead Axle L/V Ratios Through No. 20 Turnout Facing Point Diverging Side Move at 40 mph

TURNOUT PERFORMANCE

Turnout Component Failures

The most important information gathered from the Turnout Experiment concerns the service life of turnout components. Table 2 lists the major component failures documented at all turnouts on the HTL by location and component tonnage (not program tonnage) at time of failure. Component failure, in this case, is defined as a component whose condition was such that either repair or replacement was required by the track inspector before further traffic was permitted. It is recognized that determination of component failure based on visual observation is highly subjective in nature and related to the experience and knowledge of the person making the determination.

All switch point failures occurred on switch points manufactured of 300 HB rail and were all related to fatigue cracks of 4 inches to 10 inches at or near the point of switch. All three failures were related to observable metal flow along the gage side of the stock rail which caused a poor fit of the point to the stock rail. In all cases, the cracks were removed by grinding and the points restored by buildup welding of the point in track. Figure 5 shows a a typical point failure.

Frog casting failures resulted from cracks of 4 inches to 6 inches long in the manganese insert. All casting defects were first observed as a flattening or crushing of the insert surface. Figure 6 shows the void left after removal of the crack in the standard No. 20 frog facing point turnout.

Perhaps the most significant failures in terms of axle load comparison were the wing rails on the No. 10 frogs. As shown in Table 2, three failures were recorded, two of which occurred at the facing point frog after relatively short service lives and the other occurring at the trailing point frog after a moderate service life of 95 MGT. All three failures were identical: a transverse fracture of the rail at the bolt hole

Table 2. Summary of Turnout Component Failures

COMPONENT	LOCATION	TONNAGE (MGT)
Wing Rail	No. 10 Facing Point Turnout	24
Diverging Switch Point	No. 20 Trailing Point Turnout	30*
Straight Switch Point	No. 20 Facing Point Turnout	32
Wing Rail	No. 10 Facing Point Turnout	14
Diverging Switch Point	No. 20 Trailing Point Turnout	42
Frog Casting	No. 20 Facing Point Turnout	70
Frog Casting and Wing Rail	No. 20 Trailing Point Turnout	75*
Wing Rail	No. 10 Trailing Point Turnout	95
Frog Casting	No. 10 Facing Point Turnout	98

* Components had been exposed to 160 MGT of 33-ton axle load traffic prior to the HAL tonnage shown in the table

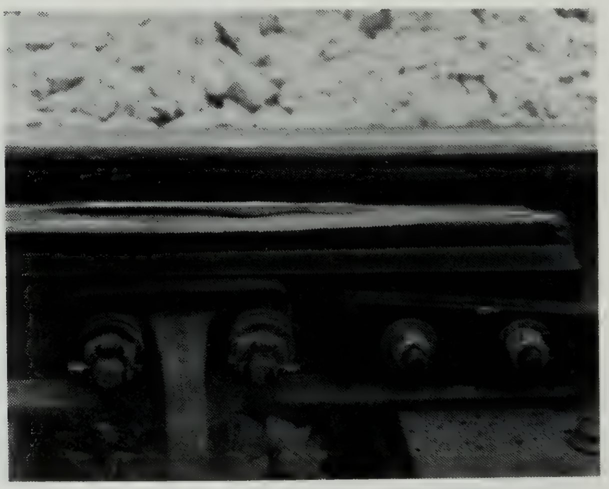

Figure 5. Typical Switch Point Failure Diverging Side of Trailing Point Turnout

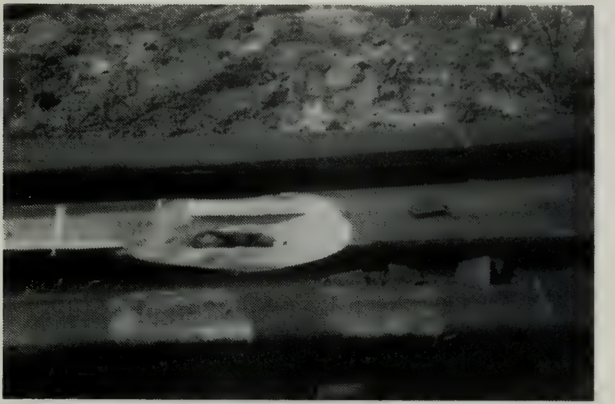

Figure 6. Void in Frog Casting after Removal of Casting Defect

located just ahead of the casting throat. What makes these failures significant in terms of axle load comparison is that similar fractures were not noted during the 1,000 MGT generated during the 33-ton axle load phase.

Figures 7 and 8 are photographs of the second wing rail failure (facing point No. 10 after 14 MGT on the wing rail), taken with the frog in track before initiation of repairs. In Figure 9, the progression of the same fracture can be seen after the rail has been removed from the frog.

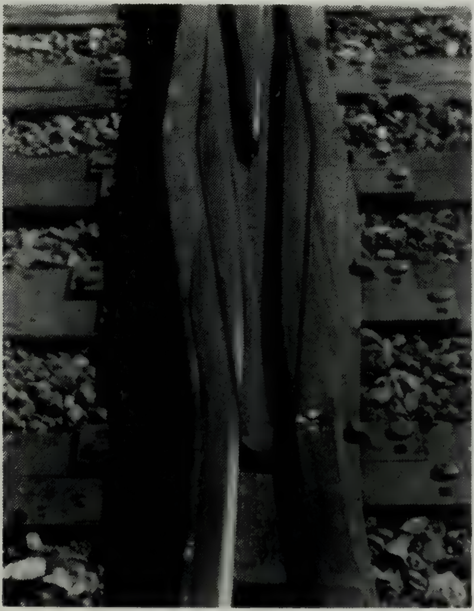

Figure 7. Location of Wing Rail Fracture on the Frog

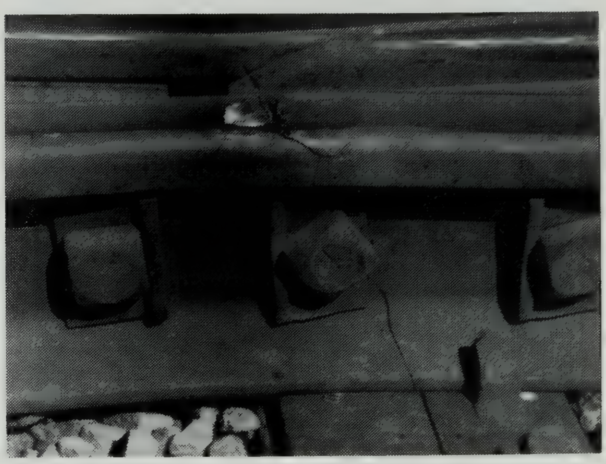

Figure 8. Side View of the Fracture Showing the Position of the Bolt Hole

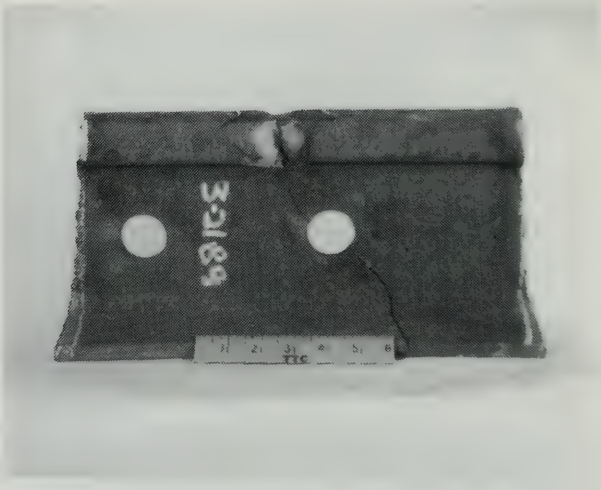

Figure 9. View of the Fracture with Wing Rail Removed

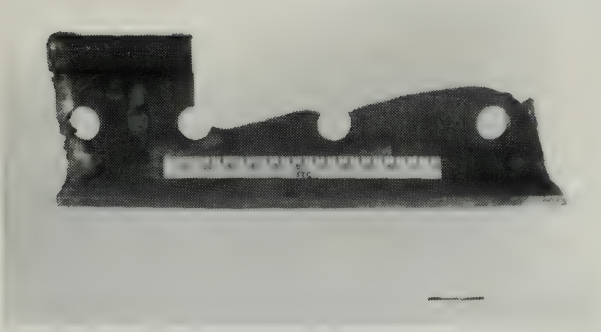

Figure 10. Longitudinal Crack through the Rail Web

Although all three defects were similar, the third failure at the trailing point frog was the most serious. Figure 10 shows how the crack progressed longitudinally in the rail web for a distance of about 16 inches before final rupture along the vertical axis.

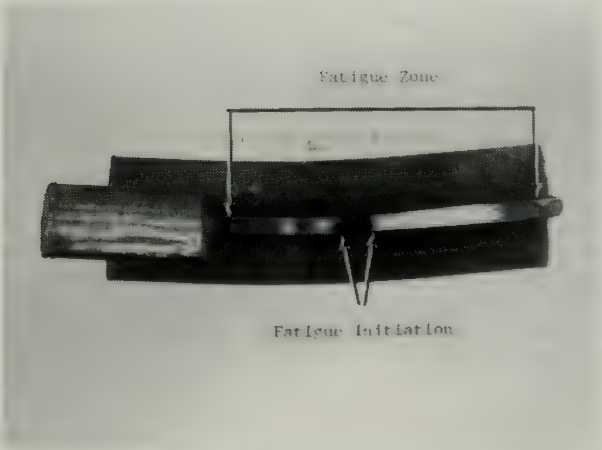

Figure 11. Top View of Wing Rail Failure Showing Fatigue Initiation and Length of Fatigue Zone

All three failures were analyzed at the TTC's Metallurgy Laboratory to determine the origin and cause of the cracks. In each case, the analysis showed the crack originated at the bolt hole on the side of the bend nearest the frog's toe and at the gage side of the rail where the surface is in tension from the bending (Figures 11 and 12). With the use of a stereo microscope and magnifying glasses, it was determined that fatigue cracks originated at the sharp edges at the periphery of the bolt holes.[3] It was also noted, during the examination, that the bolt holes responsible for the failures were only partially chamfered, resulting in sharp edges — ideal stress risers for cracks to initiate. Evidence of scuffing on the inner surface of the bolt hole, indicated the third failure (the specimen with the longitudinal crack) was also aggravated by a loose bolt.

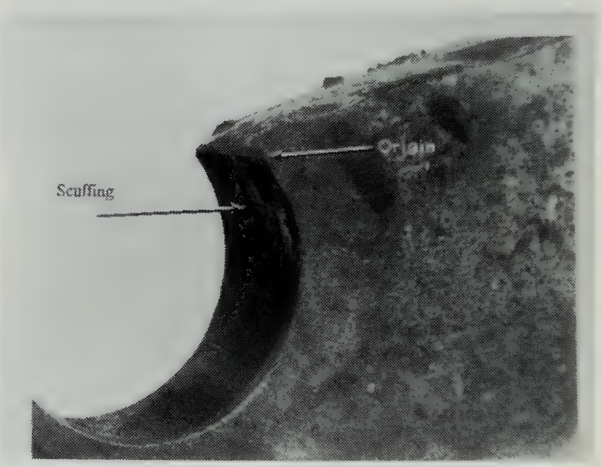

Figure 12. Close-up of Bolt Hole Showing Origin of Fracture

Turnout Maintenance Demand

The labor hours required to maintain the standard and premium component No. 20 turnouts were compared for the first 60 MGT on each turnout. As shown in Figure 13, the premium materials required about 77 percent less maintenance effort than the standard components. During the initial 60 MGT of operation, the standard turnout required repair of a cracked switch point, grinding to remove metal flow on the stock rails, grinding to reshape the manganese casting due to metal flow, and grinding to remove metal flow and surface defects on the wing and heel rails of the frog. Maintenance of the premium turnout included grinding of metal flow on the straight switch point and stock rail at 12.5 MGT, grinding of metal flow on the manganese casting at 16 MGT, and replacement of four fractured rail fasteners.

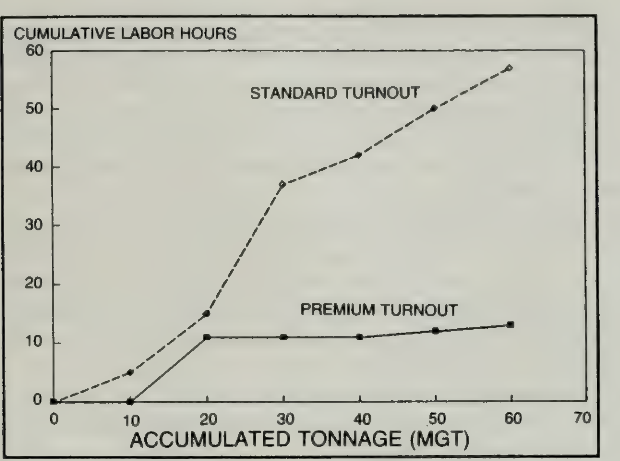

Figure 13. Comparison of Premium and Standard Component Turnout Maintenance Hours

SUMMARY OF TEST RESULTS

The results of the Turnout Experiment after 160 MGT of HAL traffic are as follows:

1. Three wing rails installed on No. 10 frogs failed as a result of fatigue cracks originating at the bolt hole near the throat of the frog, where the rail is bent to fit around the frog casting. Analysis of the cracks indicated that in each case the origin was at the edge of the hole on the field side of the rail where the rail surface is in tension from the bend. The edges of the holes were found to be sharp, presumably due to inadequate chamfering during the manufacturing process, which created stress risers. No similar fractures were noted during the 33-ton axle load program on frogs at the same locations.

2. Three AREA design switch points manufactured of 300 Brinell hardness rail exhibited fatigue cracks near the point of switch. The cracks were associated with metal flow along the gage side of the stock rail which affected the fit of the point to the stock rail.

3. The thin walled frog casting installed at the facing point No. 20 turnout failed after 70 MGT of service and the heavy wall design at the trailing point No. 10 turnout failed after 98 MGT. A third casting failure was recorded at the trailing point No. 20 after 160 MGT of 33-ton axle load traffic and 75 MGT of 39-ton axle load traffic. All three failures resulted from cracks which developed in the casting body.

4. Comparing the maintenance effort after 60 MGT of traffic, the premium component No. 20 turnout required 77 percent less maintenance than the standard component turnout installed earlier at the same location. In addition to repair of a cracked switch point, grinding of rail surface defects, such as head checking and spalling, which were observed on the 300 HB rails starting at 20 MGT, accounted for most of the increase.

REFERENCES

1. Reiff, R. P. "Heavy Haul Workshop FAST/HAL Open House," *Pre-Conference Proceedings.* Workshop on Heavy Axle Loads at the Sangre de Cristo Arts & Conference Center and the Transportation Test Center, Pueblo, Colorado, October 14-17, 1990.

2. Handal, Stephen N., "Modeling of Improvements in Turnout Performance," *Paper to be Presented at ASME/AREA Conference,* Jacksonville, Florida, 1990.

3. Brave, Glenn, "Examination of Failed Wing Rails, " *FAST/HTL Program Quarterly Progress Report No. 11,* April 1990.

FAST/HAL RAIL WELD PERFORMANCE

By: G. H. Brave*

INTRODUCTION

In the past decade, the railroad industry in the United States has seen electric flash butt welding refined to a point where it has become the standard practice for welding rails into long continuous strands. As is commonly known, in standard carbon rails the mechanical strength of a flash butt weld will generally approach that of the rail base metal. Alloy rails can now be welded with a great deal of confidence whereas 10 years ago alloy welds were not entirely dependable and on many occasions ended up as service failures. Flash butt welds in head hardened rails can now be air quenched to a hardness similar to that of the rail being welded thereby minimizing differential wear and maintaining a smoother running surface.

Flash butt welding is not always possible and rails sometimes must be joined by other means. Strands of continuous welded rails (CWR) once delivered to their field locations are most generally joined together with thermite welds. In the past, mechanical joint bars were used to join CWR but could not provide adequate stiffness required for maintaining a smooth running surface. At FAST, thermite welding is not only used to join CWR, but also becomes necessary when installing rail plugs. It is also used as a tool to replace rail fatigue defects in rails.

In the early 1970's portable flash butt welding systems were introduced to the United States and Europe as another means to perform rail welding in the field.[2] They were found to perform as well as the fixed (in-plant) flash butt welding systems.

This paper describes the fatigue performance of electric flash butt and thermite welds during 39-ton axle load testing. In determining weld performance or welded rail-end batter rates actual service MGT of each weld is used. Because of incomplete data, comparisons of 39-ton axle load to 33-ton axle loads for thermite welds can only be made to 65 MGT. Weld batter rates are available for portable flash butt welds and in-plant flash butt welds which were tested in Section 07 (5-degree curve).

TEST PROCEDURE

Thermite Welds

Only new standard thermite welds that were installed at the beginning of the 39-ton axle load lubricated testing were included in the test. These welds were installed into 5- and 6-degree curves in the high and low rails of Sections 03, 07, 25, and 31. Figure 1 is a layout of these welds and their relationship to the High Tonnage Loop (HTL). Four new alloy welds were also installed in Section 03

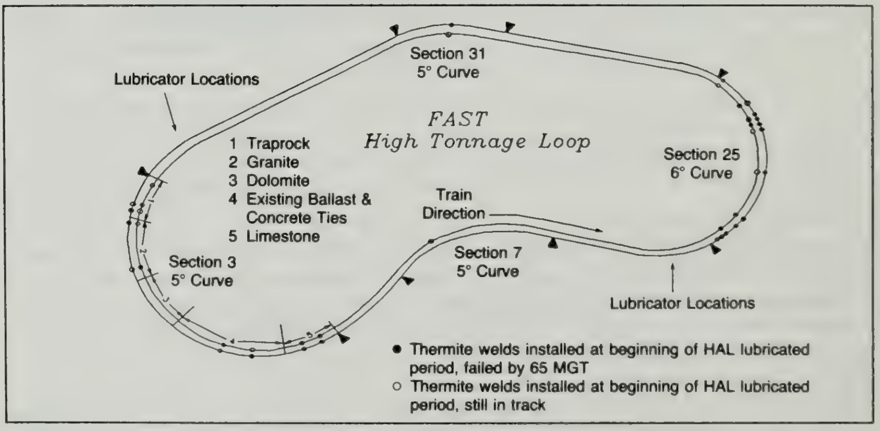
Lubricator Locations
Section 31
5° Curve
FAST
High Tonnage Loop
1 Traprock
2 Granite
3 Dolomite
4 Existing Ballast & Concrete Ties
5 Limestone
Section 25
6° Curve
Train Direction
Section 3
5° Curve
Section 7
5° Curve
Lubricator Locations
● Thermite welds installed at beginning of HAL lubricated period, failed by 65 MGT
○ Thermite welds installed at beginning of HAL lubricated period, still in track

Figure 1. Location of Thermite Welds on FAST Track

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*Engineer, Association of American Railroads and were tested independently from the standard welds. The standard welds were compared to a number of welds which were tested on the HTL under 33-ton axle loads during the Defect Occurrence and Growth Test (DOG) at FAST.[3] Because most of the test rails during the 33-ton axle load test were flash butt welded, the thermite weld population was much less than during the 39-ton axle load test. It should be mentioned that the procedures used for making the new thermite welds during the 39-ton axle load lubricated phase of the test were the same used during the 33-ton axle load test. Weld compositions were identical to those used in the 33-ton axle load test. Later into the 39-ton axle load test, new thermite welds were made using a new alignment procedure as directed by Orgo-Thermit, Inc. The height of the crown was also changed.

Electric Flash Butt Welds

There were 248 electric flash butt plant welds in Sections 03 and 25 at the start of 39-ton axle load testing. The flash butt welds joined together a variety of standard carbon 300 Bhn rails and head hardened rails. A selected number of the head hardened welds were air quenched at the request of the rail manufacturers. Many of the welds in test, which were air quenched, did not have matching non-air quenched welds to use for comparison purposes.

Portable Electric Flash Butt Welds

Portable electric flash butt welds were also tested during 39-ton axle load testing. A portable AC welder performed a number of welds in rails out of track in Section 07 (5-degree curve). Existing strands of CWR made in a plant were cut at the 19-foot 6-inch centers of the rails and portable electric flash butt welds were installed between the plant welds. Every other weld in the CWR strand was a portable flash butt weld for which direct comparisons could be made. Section 07 is traditionally a non-lubricated curve where welded rail end batter and rail wear testing is done. The low rails in Section 07, which is a reverse curve, are slightly contaminated with track grease due to the wayside track lubricator which is located on the outside of the HTL. Because of the high wear rates in Section 07, fatigue defects rarely occur in the head of test rails but still do occur in the web and base.

Operation Procedure

All welds in the test were exposed to 145 MGT of service under 39-ton axle loads. The high rail was lubricated throughout the testing period with a wayside lubricator located near the entrance to Section 25. With train operation in the counter-clockwise direction, lubrication levels were always normally higher in Section 25 as compared to Section 03. On average, the FAST train consisted of seventy-five, 125-ton hopper cars and four locomotives with an average consist weight of 12,500 tons. The average speed of the test train was 40 mph. The 39-ton axle load test train ran primarily in the counterclockwise direction for 3 MGT periods. The last 30 laps of each 3 MGT period were run in a clockwise direction at which time the lubricators were turned off for dry-down of the HTL. The dry-down was intended to clean the track for a more accurate rail flaw inspection. The 30 clockwise laps also served as a means to establish "bench marks" on the surface of existing fatigue defects in test rails and welds alike. Without the clockwise operation, fatigue defect surfaces are featureless and reveal no tell tale signs of train activities in which fatigue crack growth rates can be determined.

RESULTS AND DISCUSSION

Thermite Welds

At the beginning of 39-ton axle load lubricated operation, there were approximately 85 thermite welds located throughout the HTL. Forty of these were new welds and most were used to join individual test rails to CWR in test curves. Some of these new welds were also used to join non-test rails during 39-ton axle load tests. However, they were not used in determining weld failure rates. Some of the remaining welds had previous exposure (33-ton axle testing) and were not used for testing, therefore, had no recorded service history. Nevertheless fatigue data was collected on all welds in the HTL for purposes of generating a data base.

Table 1A. Thermite welds installed at the beginning of 39-ton axle load lubricated testing

HIGH RAIL

HTL SEC-TIE	KIT TYPE	RAIL METALLURGY	RAIL TEMP	FAILURE MODE	WELD MGT
03-0269	132/136	136 STD/132 STD	72	Detail Fracture	81.7
03-0316	136	132 STD/132 STD	67	Web Crack	41.2
03-0362	132/136	132 STD/136 STD	66	Base Crack	36.3
03-0743	136	132 HH/132 HH	50	Shelled	143.9
03-1536**	132 ALLOY	132 HH/132 HH	65	Web Crack	12.9
03-1674	136	136 HH/136 HH	42	Shelled	36.3
03-1859**	132 ALLOY	132 HH/132 HH	65	Web Crack	10.6
03-1995	136	132 HH/132 HH	-	Web Crack	45.2
07-0170	136	132 HH/132 HH	67	Web Crack	36.2
25-0046	136	132 STD/132 STD	45	Shelled	71.2
25-0070	132/136	132 STD/136 STD	55	Shelled	71.2
25-0093	132/136	136 STD/132 STD	58	Shelled	64.1
25-0182	136	132 STD/132 AHH	54	Shelled	29.2
25-0320	136	136 HH/140 STD	40	Web Crack	76.4
25-0726	136	133 STD/133 STD	52	Shelled	50.1
25-1134	132/136	132 STD/136 HH	39	Web Crack	22.4
25-1178	132/136	136 HH/132 STD	40	Web Crack	56.4
25-1225	136	132 STD/132 STD	60	Shelled	58.8
25-1249	132/136	132 STD/136 STD	66	Shelled	58.8
25-1332	136	136 HH/136 STD	40		143.9
25-1618	136	132 HH/132 FHT	72	Shelled	55.5
31-0251	136	132 STD/132 HH	87	Detail Fracture	56.2

22 welds were installed in high rail of curves, 15 failed by 65 HAL MGT, and 21 were removed or had defects by 144 MGT.

*Weld was removed because of rail maintenance.
**Alloy welds were not included in failure rate calculations.

The new welds mentioned above were monitored from the start of lubricated operation for fatigue performance evaluation. Figure 1 illustrates the location of the 40 new test welds in the HTL at the start of 39-ton axle load testing, while Tables 1A and 1B list the weld locations along with other information including: weld kit mold size and type (standard portion unless otherwise noted), rail section and metallurgy, rail temperature taken during the welding operation, and the type and MGT of the failure, if applicable. The table is divided in two parts: (1A) welds installed in high rail of curves, and (1B) welds installed in low rails of curves.

Twenty-two of the initial 40 welds were installed in 5- and 6-degree curves in the high rails. Fifteen of these initial welds failed by 65 MGT. Nineteen of the original welds in the high rails were removed or had defects by 145 MGT. The dominant modes of failure were web cracks and shelling. Although the shell defects did not cause catastrophic fracturing of the welds, they did require a cessation of operation of the FAST train to allow time for repair or removal of the weld. Repair of shelled welds usually involved 1-2 hours (depending on the severity of the shell) using electric arc weld repair. Usually immediate response was taken in the repair or removal of shells found in welds as rapid deterioration of the gage side of the rail head would occur under heavy axle loads. Of the 18 original welds installed in the low rails, three failed by 65 MGT. Sixteen were removed or had defects by 145 MGT. Again, web cracks and shelling failures were the primary cause for removal.

Thermite welds tested during 33-ton axle load testing were exposed to 65 MGT of testing. Weld records are not available beyond 65 MGT for 33-ton axle load testing but comparisons can be made

Table 1B. Thermite welds installed at the beginning of 39-ton axle load lubricated testing

LOW RAIL

HTL SEC-TIE	KIT TYPE	RAIL METALLURGY	RAIL TEMP	FAILURE MODE	WELD MGT
03-0079	132/136	133 STD/136 STD	-	Detail Fracture	143.9
03-0268	132/136	136 STD/132 STD	77	Shelled	143.9
03-0315	136	132 STD/132 STD	79		143.9
03-0363	132/136	132 STD/136 STD	75	VSH, Shelled	143.9
03-0745	136	132 HH/132 HH	-	Batter	48.8
03-1537**	132 ALLOY	132 HH/132 HH	65	Web Crack	22.4
03-1677*	136	136 HH/136 HH	-		84.4
03-1858**	132 ALLOY	132 HH/132 HH	72	Web Crack	94.9
03-1998	136	132 HH/132 HH	-	Web Crack	80.4
25-0101	136	132 STD/132 STD	52		143.9
25-0343	136	136 HH/140 STD	72	Shelled	143.9
25-0751	136	133 STD/133 STD	63	Web Crack	94.5
25-1161	132/136	132 STD/136 HH	-	Detail Fracture	143.9
25-1206	132/136	136 HH/132 STD	33	Battered	65.0
25-1255	136	132 STD/132 STD	44	Web Crack	94.9
25-1364	136	136 HH/136 STD	54	Shelled	143.9
25-1653	136	132 HH/132 FHT	40		143.9
31-0265	136	132 STD/132 HH	89	Shelled	56.2

18 welds installed in low rail of curves, 3 failed by 65 HAL MGT, and 14 were removed or had defects by 144 MGT.

*Weld was removed because of rail maintenance.

**Alloy welds were not included in failure rate calculations.

with 39-ton axle load up to this time. Table 2 lists welds that were installed at the start of 33-ton testing and also gives the million gross tons at which the welds failed. These welds were all standard mix welds. One of the four high rail welds failed before 65 MGT in Section 25, while in Section 03, one out of three failed. No welds failed in the low rails of Sections 03 or 25.

Electric Flash Butt Welds

Approximately 195 new electric flash butt test welds were installed at the start of the 39-ton axle load lubricated testing. These welds were located in Section 03 (5-degree curve) and Section 25 (6-degree curve). Layouts illustrating the locations and rail metallurgies that the welds joined can be seen in Figures 2 and 3. At the end of 145 MGT of testing, five welds failed in the high rails of Section 03 and 25, while six failed in the low rails in the same sections. Horizontal web cracks accounted for the bulk of weld failures. Table 3 lists the types of failures and the million gross tons at which the weld failures occurred. Flash butt weld failures during the 33-ton axle load test can be seen in Table 4. A total of 176 flash butt welds were tested in Sections 03 and 25 at the start of the 33-ton axle load test. These welds were exposed to 160 MGT of lubricated testing.

Table 2. Section 25 and 03 - Thermite Welds Installed at the Beginning of 33-Ton Axle Load Lubricated Testing.

HIGH RAIL

HTL SEC-TIE	RAIL METALLURGY	FAILURE MODE	WELD MGT
03-0702	132 STD/136 STD		
03-0820	136 STD/136 STD	Shelled	65.0
03-0967	136 STD/132 STD		
25-0340	136 STD/136 STD		
25-0679	136 HH/136 STD		
25-0846	136 STD/136 STD	Shelled	56.0
25-1562	136 HH/136 STD		

7 Welds installed in high rail of curves; 2 failed by 65 MGT.

LOW RAIL

SEC-TIE	RAIL METALLURGY	FAILURE MODE	WELD MGT
03-0700	132 STD/136 STD		
03-1150	136 HH/132 STD		
25-0098	132 STD/136 STD		
25-0328	136 STD/136 STD		
25-0846	136 STD/136 STD		
25-1616	136 STD/136 STD		

6 welds installed in low rail of curves; 0 failed by 65 MGT.

Portable Electric Flash Butt Welds

Seven portable electric flash butt welds along with 13 flash butt shop welds were tested in the 5-degree curve of Section 07. Six of the portable welds were mixed together in CWR strands along with a number of shop welds in standard 300 Bhn and induction head hardened rails. The welds and rail metallurgies are listed in Tables 5A and 5B. The tables are divided into two sections: The top half lists ten welds that were installed during 33-ton axle load testing and were carried over to the 39-ton axle test. The bottom half of the table lists the portable and shop welds that were installed into Section 07 during the 39-ton axle load test. The 10 shop welds had previous service exposure (55.5 MGT) during the 33-ton axle load test. The CWR strands containing the portable and shop welds were installed at 75 MGT of 39-ton axle load testing and currently have accumulated 67 MGT of testing. Some of the welds were air quenched and are designated with an asterisk in the tables. Welds tested in the reverse curve in Section 07 are primarily measured for their resistance to welded rail end batter as the curve is basically dry and free of lubrication. Fatigue data was also collected for the welds in Section 07. The low alloy in-line head hardened welds (1500 series) were installed at the start of 39-ton axle load testing and remained throughout the entire test. One of the low alloy welds (1506), in the high rail, developed a horizontal web crack and was removed at 136 MGT. Overall, the ten welds, which had previous 33-ton

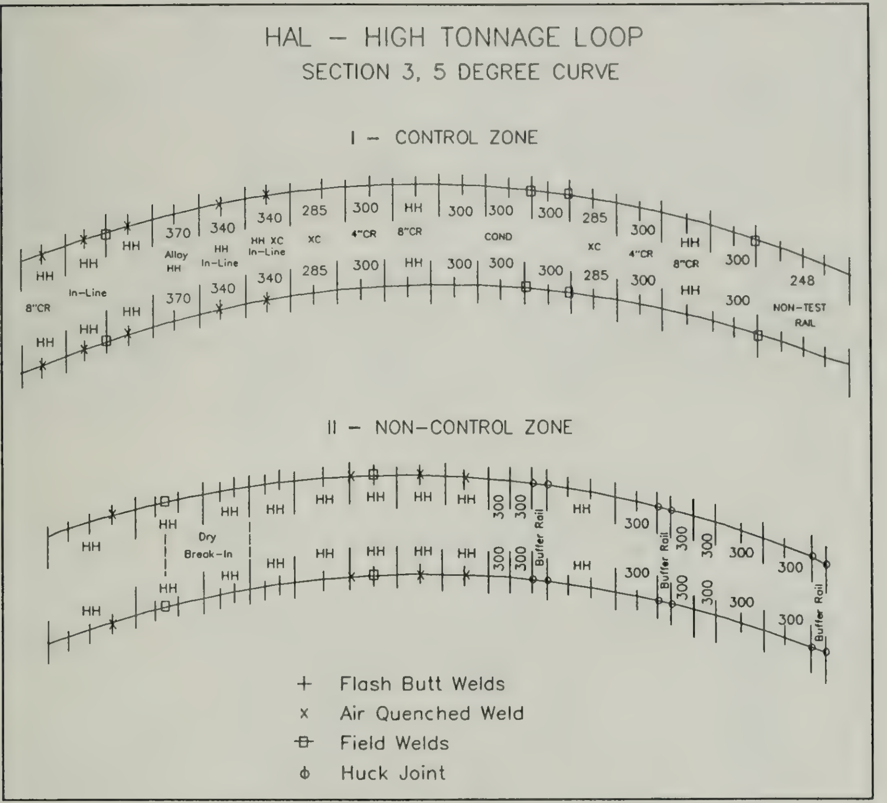

Figure 2. Weld Locations and Rail Metallurgies

axle load exposure, tend to exhibit higher batter rates than the welds installed during 39-ton axle load testing. This is probably due to accelerated batter which occurred in the welds because of the existing batter which the welds had previously developed during 33-ton axle load testing. The MGT columns in Tables 5A and 5B list only tonnage the welds accumulated during 39-ton axle load testing. This represents only the batter rate which occurred during 39-ton axle load testing. Batter rates for the welds tested during 33- and 39-ton axle load tests are calculated using only the MGT accumulated during 39-ton axle load testing. This represents only the batter rate which occurred during 39-ton axle load testing. Batter rates are determined by dividing the batter depth by each weld's service MGT.

Data in Table 5A suggest portable flash butt welds in standard carbon rail battered at rates slightly less than that of shop welds. The data also shows post weld air quenching is beneficial in welds in the induction head hardened high rails. The non-air quenched welds battered at rates two times higher than welds that were air quenched. Non-air quenched low alloy head hardened flash butt welds batter at rates similar to that of welds in standard carbon head hardened rails which were air quenched. Low alloy head hardened (AHH) 370 Bhn and standard head hardened (THH) 370 Bhn portable welds batter at similar rates (.0003 in. per MGT) in high rails.

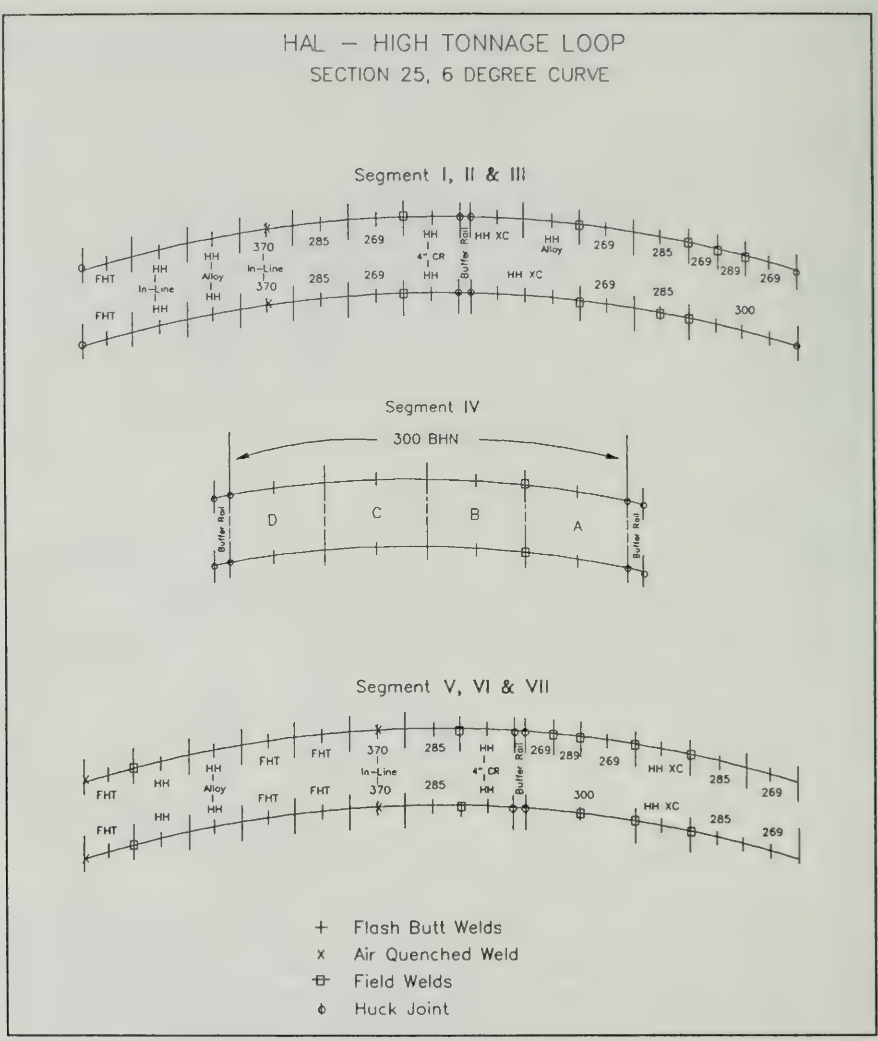

Figure 3. Weld Locations and Rail Metallurgies

In the low rail welds for both standard 300 Bhn and standard carbon head hardened and THH 370 head hardened rails batter rates were similar (approximately .0001"/MGT). Low rail portable welds in induction head hardened and THH 370 rails showed no benefits from air quenching. Low alloy AHH 370 and low alloy HH (in-line) welds showed minimal batter rates after 67 MGT of service. In the low rails, portable and shop welds that were air quenched show no benefits from air quenching.

Table 3. Failed Plant Welds 39-Ton Axle Load

HIGH RAIL

SEC-TIE	RAIL METALLURGY	FAILURE MODE	WELD MGT
03-0624	136 HH 340/136 HH 340	VERT SPLIT HEAD	140.6
03-0791	132 HH/136 HH	WEB CRACK	55.5
03-1516	132 HH/132 HH	WEB CRACK	99.3
25-1476	136 FHT/132 FHT	WEB CRACK	81.7
25-1548	132 AHH/132 AHH	WEB CRACK	121.3

100 Welds installed in high rail, 5 failed by 145 MGT.

LOW RAIL

SEC-TIE	RAIL METALLURGY	FAILURE MODE	WELD MGT
03-0698	136 HH 340/132 HH	TD	103.2
03-1058	132 STD/132 STD	WEB CRACK	100.5
25-0053	132 HH 300/132 HH 300	UNKNOWN	91.2
25-1509	136 FHT/132 FHT	TD 20%	45.2
25-1533	132 FHT/132 FHT	TD 22%	138.2
25-1558	132 FHT/132 AHH	WEB CRACK	85.5

95 Welds installed in low rail, 6 failed by 145 MGT.

Table 4. Failed Plant Welds 33-Ton Axle Load

HIGH RAIL

SEC-TIE	RAIL METALLURGY	FAILURE MODE	WELD MGT
03-0777	136 STD/136 STD	TD 41%	
25-1583	136 STD/136 STD	TD 22%	
25-0679	136 STD/136 NHH	SHELL	62.0

91 Welds installed in high rail, 3 failed by 160 MGT.

LOW RAIL

SEC-TIE	RAIL METALLURGY	FAILURE MODE	WELD MGT
03-0810	136 STD/136 STD		
03-0906	132 STD/132 STD	WEB CRACK	94.0
25-0473	136 STD/132 STD	BATTERED	56.0

84 Welds installed in low rail, 3 failed by 160 MGT.

Table 5A. Electric Flash Butt Welds in Section 07, 5-Degree Curve

HIGH RAIL

METALLURGY	TEST LOCATION	WELD TYPE	TOTAL MGT	BATTER DEPTH	BATTER RATE (IN.)
INDUCTION HH	1113	SHOP	55.51		0.0006
	1115	SHOP	55.51		0.0008
	1117	SHOP*	55.51		0.0003
	1121	SHOP*	55.51		0.0002
370 DHH	1309	SHOP	55.51		0.0003
	1311	SHOP	55.51		0.0001
370S DHH	1409	SHOP	100.00	0.0160	0.0002
	1414	SHOP	100.00	0.0310	0.0003
LOW ALLOY HH (IN-LINE)	1502	SHOP	139.00	0.0330	0.0002
	1506	SHOP*	136.00	0.0020	0.00001
300 Bhn	1705	PORTABLE	67.40	0.0000 **	
	1711	SHOP	67.40	0.0350	0.0005
INDUCTION HH	1805	SHOP	67.40	0.0240	0.0004
	1809	PORTABLE	67.40	0.0160	0.0002
	1813	SHOP	67.40	0.0220	0.0003
	1817	PORTABLE*	67.40	0.0040	0.0001
THH 370	1905	PORTABLE	67.40	0.0180	0.0003
	1911	PORTABLE*	67.40	0.0140	0.0002
LOW ALLOY AHH 370 (OFF-LINE)	2005	PORTABLE	67.40	0.0270	0.0004
	2011	PORTABLE	67.40	0.0140	0.0002

* - AIR QUENCHED WELD
**- INVALID BECAUSE OF METAL FLOW

Table 5B. Electric Flash Butt Welds in Section 07, 5-Degree Curve

LOW RAIL

METALLURGY	TEST LOCATION	WELD TYPE	TOTAL MGT	BATTER DEPTH	BATTER RATE (IN.)
INDUCTION HH	1116	SHOP	56.41		0.0002
	1118	SHOP	56.41		0.0001
	1119	SHOP*	56.41		0.0001
	1123	SHOP*	56.41		0.0002
370 DHH	1310	SHOP	56.41		0.0000
	1312	SHOP	48.81		0.0002
370S DHH	1412	SHOP	100.00	0.0160	0.0002
	1415	SHOP	100.00	0.0100	0.0001
LOW ALLOY HH (IN-LINE)	1504	SHOP	139.00	0.0060	0.00004
	1508	SHOP*	139.00	0.0000	0.0000
300 Bhn	1704	PORTABLE	67.40	0.0040	0.0001
	1710	SHOP	67.40	0.0040	0.0001
INDUCTION HH	1804	SHOP	67.40	0.0060	0.0001
	1808	PORTABLE	67.40	0.0040	0.0001
	1812	SHOP	67.40	0.0100	0.0002
	1816	PORTABLE*	67.40	0.0060	0.0001
THH 370	1904	PORTABLE	67.40	0.0040	0.0001
	1910	PORTABLE*	67.40	0.0100	0.0001
LOW ALLOY AHH 370 (OFF-LINE)	2004	PORTABLE	67.40	0.0000	0.0000
	2010	PORTABLE	67.40	0.0020	0.00003

* - AIR QUENCHED WELD

CONCLUSIONS

Thermite Welds

- During 33-ton axle load testing, 29 percent of welds installed in the high rails failed by 65 MGT. There were no failures in the low rails of Sections 03 and 25.
- During 39-ton axle load testing, the total high rail failure rate was identical for Sections 03 and 25 (67 percent), while the low rail failure rate was nearly the same (approximately 13 percent).
- For both Sections 03 and 25 (high rail thermite welds), increasing the axle loads by 20 percent (i.e. from 33-ton to 39-ton axle load) increased the failure rate by 130 percent at 65 MGT of testing.
- At 82 MGT of 39-ton axle load testing, 83 percent of thermite welds in the high rail of Section 03 were removed, while in Section 25, 92 percent of the thermite welds failed.
- As of 144 MGT, failure rates in the low rail welds were nearly the same for Sections 03 and 25 (75 percent).

Electric Flash Butt Shop Weld

- During 33-ton axle load testing, the total high rail failure rate was identical for the 5- and 6-degree curves (3 percent), while the low rail failure rate in the 5-degree curve was 8 percent and 2 percent for the 6-degree curve.
- During 39-ton axle load testing, high rail failure rates were identical for 5- and 6-degree curves (5 percent) while the low rail failure rate was 9 percent for the 6-degree curve and 4 percent for the 5-degree curve.

— For both 5- and 6-degree curves (high rail welds), increasing the axle load by 20 percent increased the failure rate 67 percent.

— In 5-degree curves (low rail welds) failure rates were 8 percent for 33-ton axle load and 4 percent for 39-ton axle loads, while in 6-degree curves a failure rate increase of 350 percent occurred from 33-ton to 39-ton axle loads.

— Horizontal web cracks were the dominant mode of failures for plant welds in 39-ton axle load testing, while both transverse defects and horizontal web cracks were responsible for plant weld failures during 33-ton testing.

Portable Electric Flash Butt Welds

Because of the limited number of portable welds in track during 39-ton axle load testing, firm conclusions cannot be derived from such a low population of test samples and trends can only be observed after 67 MGT of service.

— Portable welds in standard induction head hardened high rails batter at rates less than that of shop welds.

— Low rail welds made with portable and shop flash butt welds batter at nearly identical rates.

— In induction head hardened high rails, non-air quenched portable welds display batter rates two times higher than welds which were air quenched.

— Low rail portable welds show no benefits from post weld air quenching.

REFERENCES

1. Reiff, R. P. "Heavy Haul Workshop, FAST/HAL Open House," *Pre-Conference Proceedings*, Workshop on Heavy Axle Loads at the Sangre de Cristo Arts and Conference Center and the Transportation Test Center, Pueblo, Colorado, October 14-17, 1990.

2. American Railway Engineering Association Advisory Committee. "Rail and Trackwork Welding," Chapt. 15. *The Track Cyclopedia*. 10th Ed., Simmons-Boardman, Omaha, NE, 1985.

3. Brave, G. and Hannafious, J. "Field Welding at FAST," Quarterly Report #.033.

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FIRST REPORT
RAILWAY APPLICATIONS OF MECHANICALLY STABILIZED EMBANKMENTS USING METALLIC REINFORCEMENTS

Special thanks to Subcommittee 1 Chairman D. J. Maxey and associate member K. A. Lindamood for their contributions to this report.

> ". . . it must be conceded that in conformity with the high standards of the Pennsylvania R. R. Co., that we did right in adopting stone. Had it been some other company whose railroad was not so favorably situated with reference to good stone, and limited means, I would have favored the "concrete steel" work. It is unquestionably an economical form of construction, and in certain places, can be advantageously used"*

INTRODUCTION

Earth retention and slope stabilization practices used by North American railways have changed very little in the past 50 years. Lateral support for a vertical or near vertical soil mass and surcharge load was in most cases provided through the use of reinforced concrete retaining walls or abutments. Other less commonly used techniques for constructing vertical or near vertical retaining walls and abutments include stone masonry or brick gravity walls; timber, concrete, or metal crib walls; sheet-pile walls; soldier piles and lagging, and gabions.

However, an alternate earth retention system has been developed in the last 20 years which applies the new concept of earth reinforcement and which has the potential for replacing many conventional earth retention methods. There is increasing interest among engineers involved in the design and construction of railroad retaining structures in the potential railroad applications of this new method of earth reinforcement. This interest is in large part due to the extensive and successful use of earth reinforcement systems in the highway construction sector. Although there are now many variations of this earth reinforcement system, the most commonly known and extensively used systems involve the use of metallic reinforcements placed in between layers of compacted select granular soil. The introduction of the metallic "tensile" reinforcements into the soil mass fundamentally changes the engineering properties of that soil and the lateral earth pressures, thereby allowing the construction of a vertical or near vertical wall or slope without the lateral support typically provided by a conventional reinforced concrete retaining wall.

These earth reinforcement systems are often known by their proprietary or product names. The most commonly known system is Reinforced Earth, a proprietary product. Other earth reinforcement proprietary or product names include Welded Wire Wall, Reinforced Soil Embankment, VSL Retained Earth, Mechanically Stabilized Embankment, and Georgia Stabilized Earth. Several other systems have been developed but are not named here.

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*Excerpt from a 1902 letter by Samuel Rea, fourth Vice President to A. J. Cassat, President Pennsylvania Railroad describing the emergence of a new technology for the construction of railroad structures, "reinforced concrete." After an initial period of review and skepticism, reinforced concrete technology evolved and gained acceptance, becoming the predominant technique for providing lateral support to earth retaining transportation structures.

For the purpose of further discussion in this report, the generic non-proprietary term "Mechanically Stabilized Embankment" (MSE) will be used. The term Mechanically Stabilized Embankment will be defined as the earth reinforcement/soil reinforcement process of increasing the strength of a soil mass by inserting tensile members into the soil mass. The composite material exhibits improved shear and compressive strength relative, in comparison, to unreinforced material (1, NCHRP 290). It is recommended that the term **Mechanically Stabilized Embankments** and its abbreviation **(MSE)** be adopted by the A.R.E.A. when referencing these earth retention systems in future committee reports or in the manual.

REPORT OBJECTIVES

The objectives of this report are to inform those engineers involved with the design and construction of railroad related retaining structures and slope reinforcement with the design, performance, and technology associated with Mechanically Stabilized Embankments. This assignment was recommended by Committee 1 to the A.R.E.A. Board of Direction and was approved as a Committee 1 assignment. Acceptance of the report by the full Committee and publication as information in the A.R.E.A. Bulletin will complete Assignment D1-3-88. Upon completion of this report, this assignment will become the charge of Committee 8, Concrete Structures and Foundations. It is anticipated that Committee 8 will develop design and construction standards for MSE structures and incorporate these standards into the A.R.E.A. Manual for Railway Engineering.

There are numerous MSE systems currently in existence which use a variety of reinforcement elements such as strips, grids, sheets, rods, and fibers. These reinforcement elements are composed of a variety of materials including metals, plastics, and fibers. There are also many variations in the facing elements. However, the large majority of MSE systems currently in existence have been constructed with metallic reinforcements and precast concrete facing panels. Accordingly, this report will only address MSE systems utilizing metallic reinforcements and precast or premanufactured facing elements. In addition, this report will not address temporary structures but only permanent, critical earth retaining structures. A critical earth retaining structure is defined as one in which failure would cause loss of life, injury, or substantial economic loss and would normally be designed and constructed under the supervision and control of a professional engineer. The design service life of the structure would generally be 50 years or greater. MSE systems which use non-metallic reinforcement elements and/or other types of facing elements besides precast concrete or premanufactured metal panels will be considered in future assignments by Committee 1.

DESCRIPTION AND USE OF MSE SYSTEMS

MSE systems have three major components: backfill, tensile reinforcement, and facing elements. The combination of the backfill and reinforcements creates a composite material that has improved tensile and compressive properties relative to the backfill material alone. The composite nature of this combination of soil and reinforcements is analogous to the combination of steel reinforcements and portland cement concrete to create reinforced concrete.

The **backfill** is the most critical element of the MSE system. Its composition and properties strongly influence the design and performance of the MSE structure. Backfill for most MSE walls normally consists of select on-site or imported granular materials. While a small number of MSE structures with metallic reinforcements have in the past been constructed with finer grained clays and silts, the overall performance of these systems has not been generally acceptable. Large deformations in the facing elements have occurred as a result of creep in the backfill material or at the soil-reinforcement interface. The deformations occurred because backfill material composed of finer grained silts and clays have time-dependent and moisture sensitive characteristics that prevent the quick and effective transfer of shear stresses to the reinforcements.

The second component of an MSE system is the **soil reinforcing elements.** Under current practice, the reinforcing elements are most often composed of metallic materials although non-metallic materials such as high strength polymer geogrids and other geosynthetic materials are beginning to be used. There are many variations in the shapes, dimensions, and other details of the metallic reinforcements. In most cases the reinforcements are categorized as being strips, bar mats, grids, or rods. The reinforcements are also further categorized according to the mechanism of stress transfer between the soil and the reinforcement. Some systems such as the smooth metal strips rely on surface friction to transfer the stress from the soil to the reinforcement. Other systems such as grids and bar mats rely mostly on passive resistance to transfer stress. In almost all cases a combination of surface friction and passive resistance is involved in the stress transfer mechanism. Common to all MSE systems, the reinforcements are placed between successive layers of compacted backfill.

The third component of an MSE system is the **facing elements**. The facing elements are the least important component of an MSE wall. The facing elements do not contribute to the overall lateral support or stability of an MSE wall, but are simply a skin for the composite reinforced soil mass. The main function of the facing elements are to prevent localized sloughing and erosion of the backfill while providing an aesthetically pleasing appearance. The choice of a facing element is based on economic, aesthetic, and/or long term durability requirements. Common facing elements include precast concrete panels, prefabricated metal sheets, and wire mesh.

Figure 1, is a schematic drawing of an MSE system identifying the major components. **Figures 2, 3, 4,** and **5** are photographs illustrating typical railway applications of MSE structures.

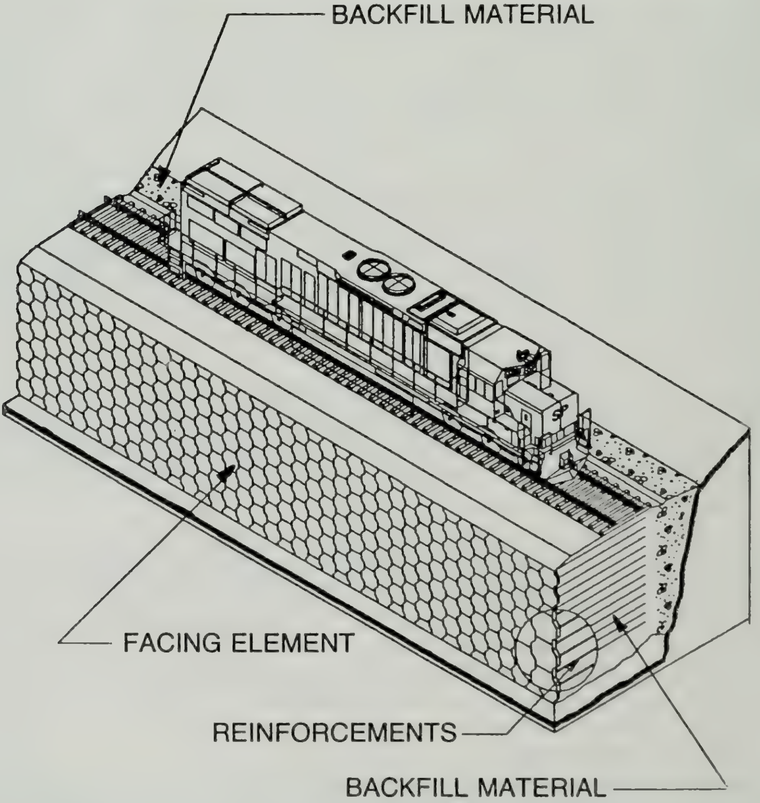


Figure 1. Diagram of a Mechanically Stabilized Embankment with Vertical Face Supporting a Railroad Live Load

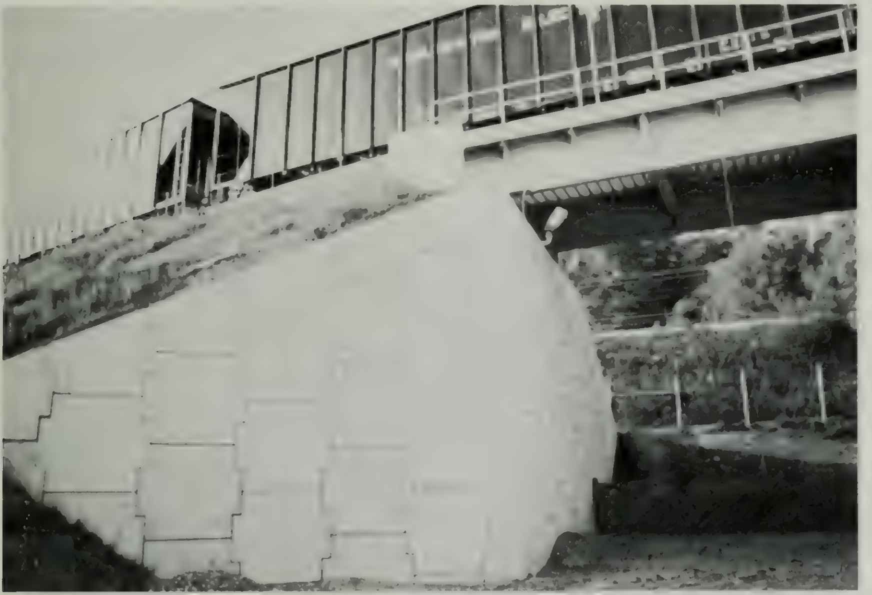


Figure 2. MSE railroad bridge abutment at the Joseph P. McNeil Electric Generating Station at Burlington, Vermont.

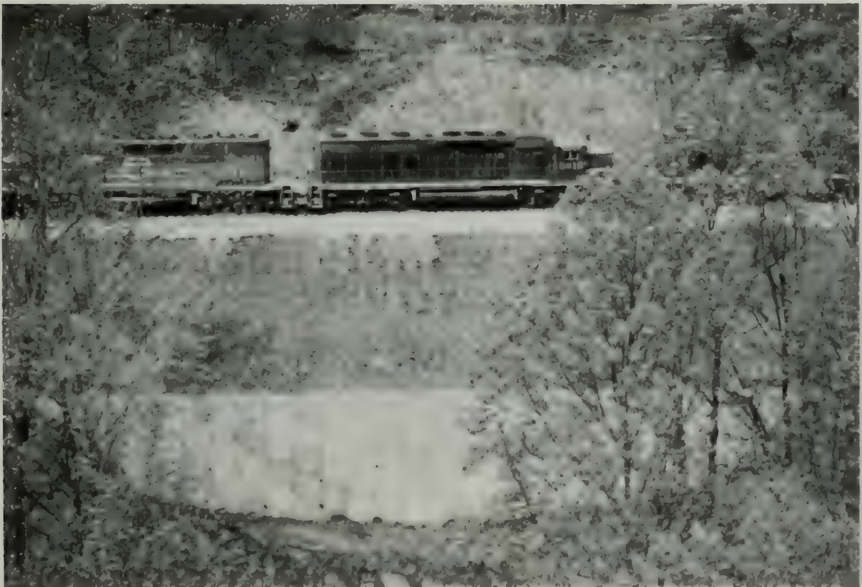


Figure 3. MSE wall on the Clinchfield Railroad in North Carolina. The MSE wall was used as part of a slide repair.

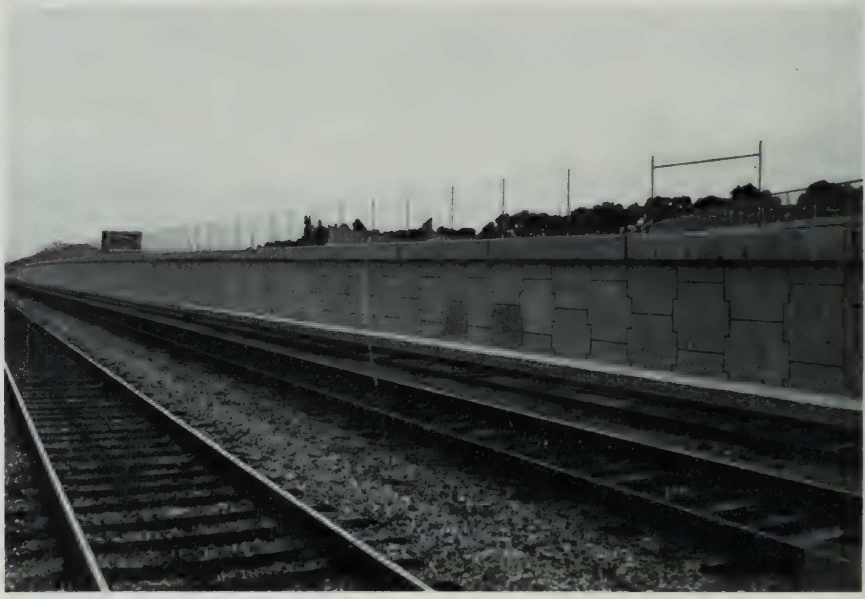


Figure 4. 2700 foot MSE wall on the Southern Pacific near Richmond, California. The wall supports a drill that is now shown in this picture.

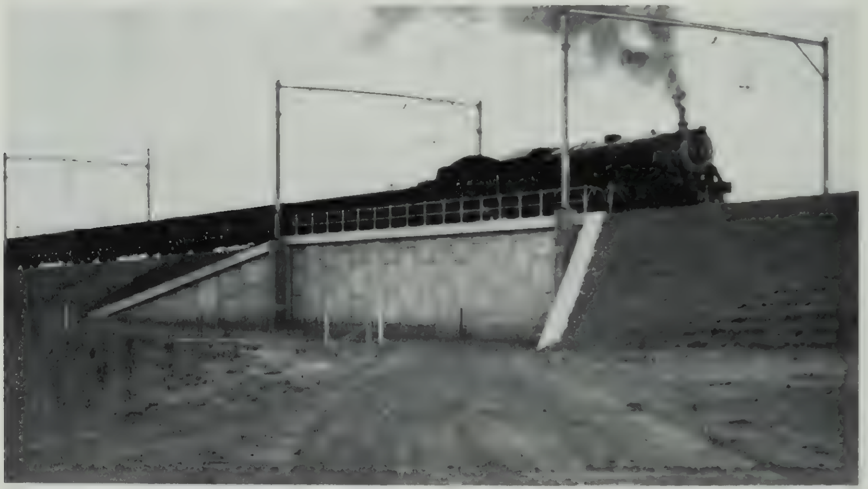


Figure 5. MSE wall in South Africa.

Many of the MSE systems that could be potentially used in railway applications have already undergone extensive analysis and testing and have been successfully used in thousands of applications throughout North America and the world. In many cases, there are certain proprietary aspects related to the use of an MSE system. However, such patent positions do not prohibit competition amongst the various systems. Typically, the MSE system and its components are designed and manufactured on a project specific basis by the proprietary supplier. The supplier will usually perform the MSE system design for the specific site conditions and designed surcharge loads and provide the reinforcement and facing materials. A general contractor will provide and place the backfill material and layers of reinforcements and assemble the facing elements under the guidance of the supplier. A thorough engineering review of the design, followed by testing and inspection of the materials and inspection of the construction techniques is typically performed by the Owner's Engineer or a Consulting Engineer acting on behalf of the Owner.

Although there are some examples of the application of tensile reinforcements dating back thousands of years, the wide-spread use and acceptance of MSE systems in the last 25 years can be traced to the efforts and inventions of the French architect/engineer Henri Vidal.

The first MSE wall was built in France in 1965, quickly followed by the construction of thousands of walls in Europe in the 1970's. The first United States wall was built in California in 1972 and the use of MSE walls within the U.S. increased somewhat after the Federal Highway Administration approved Reinforced Earth in the mid-1970's. However, it is only in the last decade that North American engineers have really begun to realize the inherent economical and technological advantages of MSE systems. The acceptance and use of MSE retaining structures is especially prevalent in highway construction and reconstruction projects. As with any emerging technology, both the practicing civil engineer and the proprietary suppliers have learned many lessons about the guidelines and limitations with respect to the design and function of MSE systems.

Several thousand MSE structures are now in service in North America, most commonly as retaining walls, but also as bridge abutments, marine structures, dams, dikes, slide buttresses, and coal and grain storage facilities. These structures currently represent nearly 30 million square feet of wall surface area with an expected annual future growth of 4 to 5 million square feet. Approximately 300,000 square feet of the above MSE structures are either adjacent to operating railroad tracks or are supporting railroad live loads.

THEORY OF MSE SYSTEMS

Mechanically Stabilized Earth systems utilizing metallic strips or grids have been extensively researched, analyzed, and validated through monitored field performance under a variety of conditions for over 20 years. When placed in granular backfill, the high strength reinforcements restrain soil deformation in the direction parallel to the reinforcements. Compressive and shear stresses in the soil are transferred to the reinforcement through either friction, passive soil pressure, or a combination of both. The magnitude of these interaction phenomena are proportional to the shear strength of the soil. Thus, the ability to transfer tensile and shear stresses to the reinforcements is directly proportional to the shear strength of the backfill. Friction and passive resistance between the soil and reinforcements and the tensile stresses generated in the reinforcements are not readily predictable by analysis alone; therefore, many pullout tests and measurements on both laboratory models and on a full scale MSE structures have been conducted to develop the semi-empirical procedures used in the design.

Within the MSE structure there is an active zone in which shear stresses in the soil are directed towards the facing of the structure. To achieve internal equilibrium, these stresses must be resisted through soil/reinforcement interaction by the reinforcements in the resistant zone of the structure, where the shear stresses are directed away from the face of the wall. The concept of active and resistant zones of an MSE system, is illustrated in **Figure 6**. An internal failure of an MSE system can occur through either loss of surface friction and/or passive failure of the reinforcement in the backfill (pullout) or tensile rupture of the reinforcements.

In summary, placing metallic reinforcements in a soil mass changes the fundamental mechanical properties and stress states so that an improved composite material is created. When properly designed to assure internal stability, the MSE mass acts as a coherent gravity structure which resists external forces through its tremendous mass.

More detailed information on MSE theory and behavior is available in the references at the end of this report.

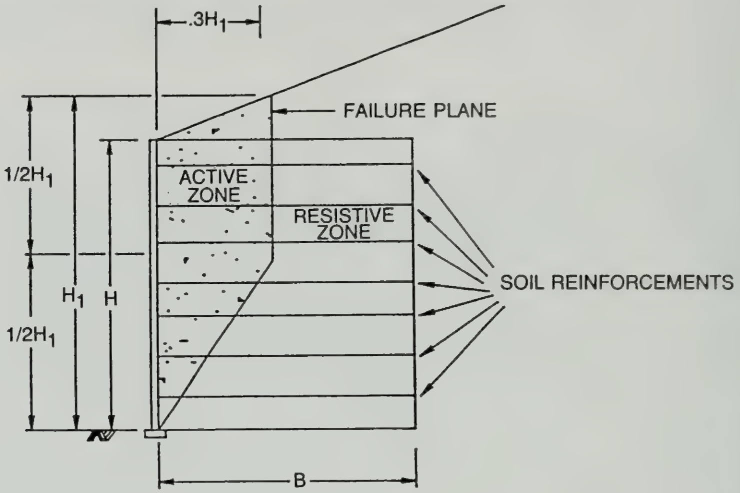

INTERNAL FAILURE PLANE OF MSE (TYPICAL)

Figure 6

EXAMPLES OF MSE SYSTEMS

Currently, the best known and most widely used MSE system is Reinforced Earth. Reinforced Earth is a proprietary system designed and supplied by the Reinforced Earth Company to whom a variety of patents covering the design, engineering, and construction have been issued. This system utilizes prefabricated galvanized ribbed metal strips as the reinforcement material. Precast concrete or premanufactured metal facing panels are fastened to the strips. Granular backfill material is used. The stress transfer mechanism for the ribbed strips is predominantly the passive resistance generated by the ribs. Surface friction along the strips also contributes to the stress transfer between the soil and the strips.

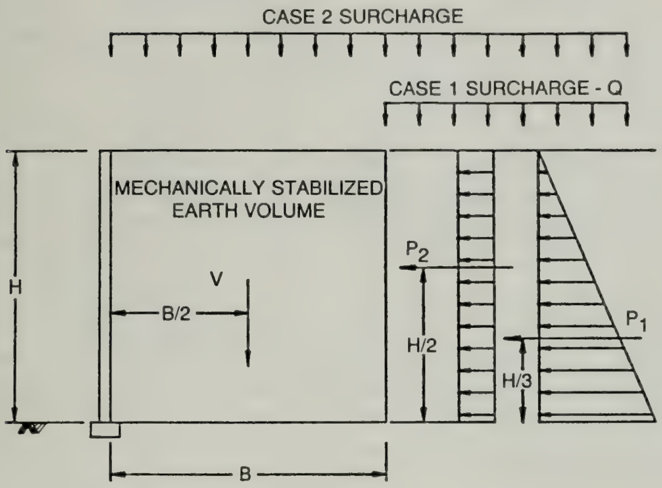
Other MSE systems similar to Reinforced Earth utilizing metallic reinforcements include VSL Retained Earth, Hilfiker Reinforced Soil Embankment, Hilfiker Welded Wire Walls, Georgia Stabilized Embankment, and the California Mechanically Stabilized Embankment. These systems use grids in the form of metallic bar mats, or welded wire meshes as the reinforcement material. The bar mats or wire meshes are in general attached to precast concrete facing panels. Granular backfill materials are usually specified to promote good drainage and void creep. Grid systems such as these, transfer most of the stress to the backfill through passive soil resistance on the transverse members of the grid or mesh. There is also a small contribution of surface friction from the horizontal surface of the grid.

Other MSE systems, which utilize geosynthetic materials such as geotextiles and high strength polymer geogrids as reinforcements have been developed or are under development. Examples of these other non-metallic reinforcement systems will be addressed in upcoming Committee 1 assignments.

DESIGN OF MSE SYSTEMS

The design of an MSE structure includes analyses of both external and internal stability. **Figures 6, 7, and 8** illustrate the typical geometry and important internal and external forces for an MSE system. An MSE wall or near vertical wall is designed as a reinforced mass that acts as a coherent gravity

EXTERNAL FORCE ACTING ON LEVEL SURCHARGE CASE (TYPICAL)



H = HEIGHT OF MSE STRUCTURE

B = LENGTH OF REINFORCEMENTS

V = MASS OF MSE VOLUME = γHB

P_1 = HORIZONTAL EARTH PRESSURE = $1/2\ H^2 \gamma K_A$

P_2 = HORIZONTAL EARTH FORCE DUE TO SURCHARGE LOADS = QHK_A

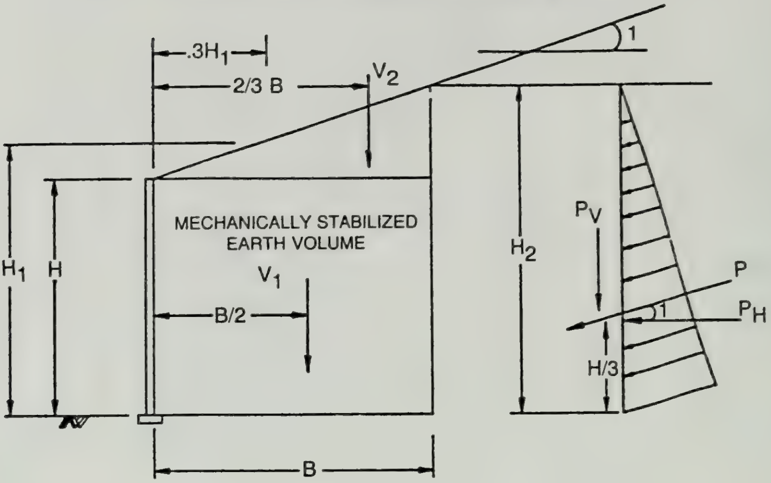
Q = CASE 1 SURCHARGE IS USED FOR DETERMINATION OF PULLOUT OF REINFORCEMENTS, SLIDING, AND OVERTURNING SAFETY FACTORS
CASE 2 SURCHARGE IS USED FOR DETERMINATION OF MAXIMUM BEARING PRESSURE AND MAXIMUM TENSILE STRESS IN REINFORCEMENTS.

$K_A = \tan^2 (45 - \phi/2)$

Figure 7

structure to resist external loads. However, to be effective, it must first be internally stable. Therefore, the internal stability of the structure is first checked to assure this performance characteristic. Then, the external stability of the coherent MSE mass is checked for conventional overturning and sliding. In addition, the foundation must be examined for settlement, bearing capacity failure,and overall global stability. In assessing the bearing capacity stability, it is important to realize that a MSE structure is a flexible system which can tolerate post construction settlements without structural distress. When compared to conventional concrete walls, the allowable differential settlement is much higher. For this reason, the allowable bearing pressure of an MSE structure is much higher than the typical rigid concrete retaining wall. This is an important factor contributing to the economic and technical advantage of MSE systems over more conventional earth retaining methods.

EXTERNAL FORCE ACTING ON SLOPING BACKFILL CASE (TYPICAL)



H = FACING HEIGHT OF MSE STRUCTURE
H_1 = THEORETICAL HEIGHT OF MSE STRUCTURE
H_2 = HEIGHT ON WHICH EARTH PRESSURE IS ACTING
B = LENGTH OF REINFORCEMENTS
P = TOTAL EARTH PRESSURE ACTING ON MSE STRUCTURE = $1/2\ H^2 \gamma K_A$
P_H = HORIZONTAL COMPONENT OF EARTH PRESSURE P x COS 1
P_V = VERTICAL COMPONENT OF EARTH PRESSURE P x SIN 1
V_1 = MASS OF MSE VOLUME = γHB
V_2 = MASS OF SLOPING BACKFILL = $1/2 \gamma B(H_2 - H)$
I = ANGLE OF INCLINATION OF SLOPING BACKFILL

$$K_A = \text{COS } 1 \left[\frac{\text{COS } 1 - \sqrt{\text{COS}^2 1 - \text{COS}^2 \phi}}{\text{COS } 1 + \sqrt{\text{COS}^2 1 - \text{COS}^2 \phi}} \right]$$

Figure 8

The internal stability analysis determines the effects of the horizontal earth pressures generated by the weight of the soil/backfill mass, lateral pressures acting on the mass, and all surcharge loads. Generally accepted practice requires that surcharge loads be considered as an equivalent uniform soil load acting on top and/or behind the mass. However, theoretical solutions which consider the soil as an elastic medium for distribution of point or line loads, may also be utilized. With respect to railroad surcharge loads, a Cooper E-80 loading is generally simulated by a uniform design soil surcharge of approximately 9 feet.

Horizontal pressures are determined at each reinforcement level and the reinforcements are then checked for both pullout adherence between the soil and reinforcing element, and tensile failure. The length, spacing, and cross sectional area of the reinforcement are calculated to provide an adequate "Factor of Safety" for each potential failure mode. A Factor of Safety of 1.5 is considered acceptable for adherence or pullout. The minimum Factor of Safety against tensile failure of the reinforcing elements is based on the standard structural steel design codes such as AASHTO or AISC. Due to the potentially catastrophic nature of a tensile rupture, the required Factor of Safety is always higher than that required for a pullout resistance and is generally set at 1.82 (0.55 Fy).

Precast facing elements are generally designed by the supplier. Earth pressures on the facing elements are low, in the range of 0.20 to 2.5 ksf. Wall panels are for the most part reinforced concrete and designed in accordance with prevailing codes and standards.

DURABILITY OF METALLIC REINFORCEMENT ELEMENTS

A common concern of all engineers considering the use of MSE structures is the long term durability of the metallic reinforcements. Most MSE systems generally specify that the high strength steel reinforcing elements be galvanized in order to inhibit corrosion. In the design process, an estimation of the corrosion rate of the strips must be considered in order to confirm an adequate design life of a structure.

Fortunately, the corrosion characteristics of steel buried in ground have been thoroughly researched and well documented. Extensive research on the durability of buried galvanized steel has been performed by the National Bureau of Standards (NBS). The NBS research consisted of a testing program lasting 45 years for a variety of soil types. The details of this corrosion investigation were published in *Circular 579*, "Underground Corrosion," by Melvin Romanoff, in April, 1957.

Figure 9 provides typical corrosion parameters related to the metallic element service life in an MSE structure.

TYPICAL CORROSION PARAMETERS:

Loss of Zinc	6 um/Year for First Two Years 2.5 um/Year for Subsequent Years
Loss of Carbon Steel	9 um/Year Per Side of Reinforcement After Depletion of Zinc

Typically at the end of design life of the structure, the reinforcements should provide adequate cross sectional area to develop $f_{allowable}$ (i.e.: 0.55 F_y or 0.50 F_u depending upon the controlling section).

Figure 9

In most cases specified granular backfill have low corrosion potential and favorable drainage characteristics which, in combination with the galvanized reinforcements, further inhibits the loss of steel due to corrosion. Based on the known characteristics of a particular backfill material, design lives of 75 to 100 years are typically specified by public authorities.

In summary, sufficient data concerning the corrosion characteristics of metallic reinforcements is available to ensure an MSE structure design life of at least 100 years.

BACKFILL FOR MSE SYSTEMS

Virtually all MSE structures constructed with metallic reinforcements have used granular soil as the backfill. Granular material is selected as backfill for the following reasons:

1. Granular soils are a much more effective backfill material for transferring shear stresses from the soil to the reinforcements than are fine-grained cohesive soils. In addition, granular soils are less prone to the detrimental time-dependent and moisture-sensitive characteristics of fine-grained cohesive soils.
2. Granular soils are relatively easy to place and compact. The performance of these soils is not significantly affected by construction and installation rates or adverse weather conditions.

3. Granular soils exhibit little or no creep. Therefore, the vertical or near vertical face of an MSE structure is not apt to deform with the passage of time or with high moisture contents in the backfill.
4. Granular soils are relatively free draining which reduces the potential corrosion of the metallic reinforcements.

The granular soil should be selected based on an analysis of its electromechanical characteristics which, in turn, will indicate the corrosion potential of the backfill.

Typical gradation and electromechanical specifications for selected granular backfill are as follows:

1. **Gradation.**

Sieve Size	*% Passing*
6″	100
3″	75-100
#200	0-15

2. **Plasticity Index not to exceed 6.**
3. **Minimum friction angle of 34 degrees at a backfill relative compaction of 95% of ASTM D-698.**
4. **Electro-Chemical Properties.**

ph	5-10
Chlorides	Less than 200 ppm
Sulfate	Less than 1000 ppm
Resistivity	Greater than 3000 ohm-cm

An essential element of the MSE construction process is a thorough quality control program related to the selection and placement of the backfill materials. The quality control for the backfill materials should include continuous inspection of the backfill and reinforcement placement, sufficient laboratory testing of the backfill soils to ensure compliance with the specifications, and frequent field density tests and inspection of construction techniques.

CONSTRUCTION OF MSE SYSTEMS

The typical construction steps for an MSE system utilizing metallic reinforcements and precast or premanufactured facing panels are as follows:

1. Site preparation including grading of the foundation area, removal of unsuitable foundation materials and replacement with select compacted backfill (if necessary), followed by proof rolling of the foundation area.
2. Construction of an unreinforced, smooth finish, concrete leveling pad along the bottom of the wall face. The leveling pad is nominally 6 inches thick and 12 inches wide.
3. Installation of the initial course of precast concrete or premanufactured steel facing units.
4. Placement and compaction of the initial layer of backfill.
5. Placement of the initial levels of reinforcements including attachment of the elements (strips, bars or mats) to the inside of the facing units.
6. Placement and compaction of successive layers of backfill over successive levels of reinforcements. Manual backfill placement and compaction techniques may be required in the area immediately adjacent to the back of the facing units.

7. Installation of successive courses of precast face units until the full height and length of the structure is completed.
8. Placement of copings, stub abutments, beams, etc. if required.

An MSE retaining wall can be constructed with relatively small earth moving equipment and small labor crews. Typically, a small crane is required to handle the precast concrete or premanufactured facing units. Dump trucks, front-end loaders and compactors are used to spread and compact the backfill. Labor crews consisting of 4 to 5 men place the reinforcements and install the facing units.

Typical production rates for the crew size and equipment stated above average between 700 to 1200 square feet of wall surface per day, fully backfilled. The production rate will vary depending on the size of the structure, accessibility of the site, and the experience and competence of the work crew and supervisors. The ease of construction and the relatively high production rates are important factors contributing to the economic advantages of MSE retaining structures when compared to conventional retaining walls.

COST OF MSE SYSTEMS

In general, the design and construction of MSE systems has developed to the stage where it is the most economic means of constructing a vertical or near vertical retaining structure or bridge abutment greater than 20 feet in height. This is especially true when compared to conventional walls constructed in areas of limited access and right-of-way and in areas which have poor foundation conditions. The main factor contributing to the low cost of an MSE system is that the largest percentage of an MSE system is composed of soil. Backfill soils, especially when available on site can be very inexpensive. In addition, the placement of the backfill is quick and easy, requiring essentially simple earthmoving techniques.

The cost of constructing an MSE wall for highway transportation structures has been found to vary from 1/2 to 2/3 of the cost of conventional walls for wall heights of 20 feet or greater. Also, the higher the wall the greater the difference between MSE retaining structures and conventional retaining structures. The cost of MSE systems has actually decreased in constant dollars because of the introduction of competitive systems and improvements in the technology.

REVIEW AND ACCEPTANCE OF MSE SYSTEMS

MSE systems have proven to be a safe and economical solution for constructing general civil and highway earth retention structures. A number of in-depth, comprehensive research programs on the mechanisms and behavior of MSE systems have been conducted by the suppliers and by state and federal agencies. Thousands of MSE systems utilizing metallic reinforcements have been constructed with very few reported design failures. Research and instrumentation has consistently concluded that the expected service life of properly constructed MSE structures will exceed the 75 to 100 year design requirements.

Most state and federal agencies and many engineering consultants have established standard policies and procedures for the design and construction of MSE systems based on the extensive research and development available to date. Because many of the MSE systems are proprietary, formal policies and procedures for the initial approval of an MSE system have been established. New systems are subject to a rigorous engineering evaluation by structural and geotechnical engineers followed by several years of experimental field tests before a system is approved for general use.

The latest version of the AASHTO-AGC-ARBTA Joint Committee, "Task Force 27 Design Guidelines for use of Inextensible Reinforcements for Mechanically Stabilized Earth Walls in Pavement Applications (Part 1)" is now available as a reference which discusses many of the design policies and guidelines that have been developed in the past few years.

EXAMPLES OF MSE RAILROAD APPLICATIONS

Application of MSE systems for railroad use should be very similar to the widespread use of MSE systems for the construction and reconstruction of highways. As previously stated, the predominant use of highway MSE structures has been for retaining walls and bridge abutments.

MSE structures used in railroad applications can be divided into two categories, those structures located adjacent to rail lines but not supporting railroad loads, and those structures actually supporting railroad loads. Within North America, there are already numerous constructed examples of the first category where highway retaining structures and bridge abutments have been built adjacent to railway lines.

Internationally, there are many examples of the second category with MSE structures supporting railroad live loads including railway lines with passenger, freight, and mixed traffic. However, use of MSE structures on North American freight lines has been limited. Examples of several MSE structures utilizing metallic reinforcements and subjected to freight railroad surcharge loading include the following:

1. An MSE retaining wall was used as part of a slide repair on the Clinchfield Railroad in North Carolina. The wall was constructed by railroad bridge and building forces. The circumstances surrounding the design and construction of this wall were reported in the 1980 A.R.E.A. Technical Conference.
2. Two MSE structures have been constructed on Conrail. One structure was a curtain wall abutment in Cleveland, Ohio and a MSE wall was built in order to eliminate a span of a timber trestle near Gallipolis, Ohio.
3. MSE abutments, 35 feet high, were used at a wood chip unloading trestle at the Joseph C. McNeil Electric Generating Station in Burlington, Vermont. The abutments are located over very poor soils and have been able to sustain 16 inches of settlement with no discernible distress to the precast concrete facing panels.
4. A 2700 foot long MSE wall with heights varying from 5 to 20 feet was designed by the California Department of Transportation in connection with the relocation of an important Southern Pacific Transportation Company industrial drill track at Richmond, California. This retaining wall was constructed over weak bay mud and recently withstood the 7.1 Richter event Loma Prieta earthquake without any damage or detrimental movement.

Significant use of MSE retaining structures has occurred on several rail transit lines including the Metropolitan Atlanta Transit Authority, Sacramento and Portland's Light Rail systems, and the Washington METRO.

SPECIAL DESIGN CONSIDERATIONS FOR RAILROAD MSE STRUCTURES

Although the design and use of MSE systems for railroads will be similar to that for highways, there are some notable differences with regard to design philosophy, static and dynamic loads, stray currents, infiltration through the roadbed, multiple use of railroad rights-of-way by utilities, and high settlements in adjacent ground and facilities.

The design requirements for railroad structures are in general more conservative than similar requirements for highways. The analysis and design requirements for MSE structures subjected to railroad loading need to be evaluated and modified to reflect the current practice contained in the A.R.E.A. Manual for the design of retaining walls and abutments. Some of the changes might require an increase in the various factors of safety and an increase in the service life from the values used by highway designers. The effect of these higher factors of safety and longer service life requirements would most probably result in an increase in the length, thickness, strength, or number of reinforcements or some combination thereof when compared to a similar highway retaining structure or abutment.

Both the static and dynamic loading on a railroad structure are significantly greater than highway loads. The increase in the static loads and its effect on the MSE structure can be readily determined and accommodated as described above, by increasing the length, thickness, strength, or number of reinforcements.

The effect of rail traffic induced vibrations on the stress transfer mechanism (friction, passive resistance, interlocking) between the backfill material and the reinforcements is not clearly known. However, research by Elias and Egan (5, TRR 1030) has indicated that a 10 to 20 percent reduction in the soil/reinforcement interaction mechanism should be considered. Also, an additional horizontal load, termed the dynamic earth pressure, should be incorporated into the design of MSE structures subjected to railroad traffic loading.

On electric traction railroads, stray currents from third rail or catenary systems and their effect on the rate of corrosion of the metallic reinforcements is a concern. Research and studies have shown that a risk of accelerated corrosion occurs only with systems utilizing direct current and cannot occur with alternating current systems. The use of MSE structures on direct current traction railroads can usually be accomplished by retaining a firm that specializes in corrosion studies to investigate and make site specific recommendations such as the use of high resistivity backfill and electrical isolation through the use of epoxy coating on the metallic reinforcements.

Another cause for concern of accelerated corrosion of the reinforcement elements results from the relatively high permeability of railroad roadbeds. The backfill and reinforcements can be exposed to precipitation and other potentially corrosive substances infiltrating through the roadbed. This potential problem can be easily corrected by incorporating an impermeable geomembrane connected to lateral drains beneath the subballast but above the level of any reinforcements.

Railroad rights-of-way are often used to install longitudinal utilities such as pipelines, cables, and fiber optic lines. The ability to install these facilities within the area of an MSE structure at some point in the future may be impaired or precluded by the presence of the upper reinforcement layers. This situation can be rectified by limiting placement of the reinforcements to a zone below the potential depth of excavation (4 to 6 feet) and/or by placing conduits to allow for future utility installations.

Finally, there has been some concern voiced that unusually high ground settlements will occur in the ground and in any facilities located adjacent to a MSE structure. The basis for this concern results from the fact that MSE structures are relatively flexible and can accommodate much higher levels of differential settlement than are normally observed with rigid, pile-supported concrete retaining structures. Therefore, the ground and any facilities located adjacent to a MSE structure may experience higher settlements than experienced with conventional retaining walls. However, these higher settlements should only be noticeable in close proximity to the structure and also for relatively high MSE structures constructed on poor foundations. Modern geotechnical investigations and analyses allows for a fairly accurate determination of the total anticipated settlement and the area over which the settlement will occur for a given MSE structure. In most cases, once this settlement information is determined, the adverse impact on the adjacent ground or facilities can be avoided or significantly lessened.

SUMMARY AND RECOMMENDATIONS

This report reviews the development and use of a new technique for improving the mechanical properties of earth which has many potential railroad applications related to the construction of retaining walls, bridge abutments, and steep embankment slopes. The mechanical properties of a soil mass are improved by the placement of layers of metallic strips, grids, bars or mats in between layers of soil. The "reinforced" soil mass is called **Mechanically Stabilized Embankments (MSE). MSE** structures have been extensively used by North American highway agencies and have proven to be a versatile, easily constructed, cost-effective method for constructing permanent vertical or near vertical walls, slopes, and bridge abutments. Some of the **MSE** systems are proprietary and are known by their proprietary names.

The theory, analysis, design, and performance of **MSE** systems have developed to the stage where the technology can be readily adapted for use on North American freight railroads. Design and construction guidelines and specifications for railroad MSE systems should be considered for incorporation into the A.R.E.A. Manual.

BIBLIOGRAPHY

1. Mitchell, J. K. and Villet, C. B. W. "Reinforcement of Earth Slopes and Embankments" National Cooperative Highway Research Program Report 290, Transportation Research Board. (June 1987).
2. "Reinforced Earth - Fifteen Years of Experience and Lessons for the Future," Reinforced Earth Company. (June 1986).
3. "California Department of Transportation, Highway Design Manual," Topic 210-Earth Retaining Systems, (January 1987).
4. Patel M. G., Shah R. N., Dash O., and Ved N. V. "Overview of Pennsylvania's General Design and Construction Specifications For Proprietary (Prefabricated) Walls" *preprint* Transportation Research Board 68th Annual Meeting (January 1989).
5. Ellias, V. and Egan, P. D. "Use of Reinforced Earth for Retained Embankments in Railroad Applications" Transportation Research Record 1030, Transportation Research Board, (September 1986).

COMMITTEE 2—TRACK MEASURING SYSTEMS

Chairman: R. F. Silbaugh

RECOMMENDATIONS FOR ESTABLISHING AND MAINTAINING A TRACK GEOMETRY DATA BASE

Subcommittee Chairman: G. W. Oberlechner

1. General

A data base is required to provide an automated method of organizing and storing track information, which is accessible and updatable. This data base provides a structure for maintaining a history of important records. It can also be used to tie into other corporate data files to provide additional information for the preparation of miscellaneous reports, estimates, studies, etc.

A data base can be defined as a collection of records organized according to keys, for simplified access, viewing, updating and sorting. Keys uniquely identify records in the data base and may include:

1. Division Name
2. Subdivision Name
3. Line Name
4. Track
5. Location (Milepost/Feet)
6. Date
7. Time

The first five of these items are essential for identifying the location of specific pieces of geometry data.

Items six and seven allow the user to identify different recording runs over the same location by date and time. This discrete identity is necessary if comparisons are to be made.

The track geometry data base should provide the user with the ability to compare analyzed data in various ways, generate track charts and reports, assist in planning track maintenance, study track degradation characteristics and establish historical and existing track conditions for accident analysis.

2. Track Geometry Data Base Organization

The geometry data base is built around the locations of selected features of track structure. In this data base, the track design features and their geometric measurements are related to these locations. The features of the data base will include, but not be limited to, the following:

A. Key identifiers
   (1) Division
   (2) Subdivision
   (3) Roadmaster
   (4) Line
   (5) Track
   (6) Location
B. Wayside events
   - Milepost
   - Signals
   - Lubricators, etc.
C. Track Structure discrepancies
   (1) Construction (wood/concrete ties, rail weight, ballast conditions, etc.)
   (2) Geometry (Bridges, Switches, Curve/Tangent, etc.)

  (3) Track Maintenance History
D. Track Speed
E. Track Class

3. Information to Store in a Geometry Data Base

A. The number of data elements is established by the number of features that are stored in the data base. These features include, but are not limited to, the items included in section 2 above.

B. Only analyzed data for each element is stored. These may include:
  (1) Exception length summary
  (2) Exception count summary
  (3) TQI
  (4) RQI
  (5) Parameter Summation
  (6) Single Exceptions
  (7) Curve Analysis

C. The following minimum parameters should be used to establish analyzed data where applicable:
  (1) Gage
  (2) Crosslevel (warp, twist)
  (3) Alignment (curvature)
  (4) Surface
  (5) Third Rail (catenary)
  (6) Corrugation
  (7) Rail Wear
  (8) Maintenance Records
  (9) Clearances
  (10) Track Modules (tie, ballast, subgrade condition)
  (11) Rail Flaws
  Items (1) through (7) inclusive should have the ability to be cross referenced to other data bases storing information pertaining to items (8) through (11).

4. Output Expected From a Geometry Data Base

The data stored in a geometry data base should enable the user to produce meaningful reports and derive accurate predictions for track maintenance programs. Depending upon the application, the data should be flexible enough to be used for graphs, charts and/or tables.

5. How Often to Update a Geometry Data Base

The geometry data base should be updated each time a new measurement data becomes available. The new data should be identified by date and time. This new data should be appended to the existing geometry data rather than be used to replace it. This will provide the user with multiple sets of data to perform comparisons.

6. How Long to Keep Geometry Data

Geometry data should be kept active for a minimum of one year, but at least until the next measurements are collected. It is recommended that old data be achieved for long term storage rather than simply erased. The physical limits of the host computer will dictate the amounts of data that can be active.

7. Compatibility Concerns

Computer hardware and software should be selected with past, present and future compatibility in mind. Establishing an entirely new data environment can render existing data useless unless communications and/or data translation concerns are carefully addressed. Likewise, future performance will suffer if new systems are restricted to fit within antiquated existing systems.

8. Computer Requirements

Computer hardware and software selected for the maintenance of a geometry data base should have the ability to deal with data files that will routinely exceed 100 megabytes in size. The operating system, application programs and the physical hardware must all share this ability for the current data and any archives or back-ups. It is further recommended that display and hard copy devices be capable of graphic output.

COMMITTEE 17—HIGH SPEED RAIL

Chairman: R. D. Johnson

Report of Subcommittee D-1-89

REVIEW OF HIGH SPEED RAIL SYSTEMS AROUND THE WORLD AND IN NORTH AMERICA

Subcommittee Chairman: G. E. Hartsoe

Committee 17 was formed to study various aspects of high speed rail passenger train operation. While Amtrak operates at speeds of 125 m.p.h., there are no rail systems in the United States that presently exceed 125 M.P.H. However, there are several high speed systems that are currently in the planning stage in the United States. Furthermore, there are a number of systems throughout the world that operate at speeds over 125 M.P.H. A number of these high speed systems are being expanded, Some are under construction, and additional ones are in the design or planning stages.

A review has been made and information compiled to summarize the status of the various high speed rail systems that presently exist, and of future plans now being considered. This information is being submitted for informational purposes for all members of the American Railway Engineering Association.

It should be recognized that the information presented herewith concerning the status of high speed rail systems is constantly changing and frequent up-dates will be necessary to maintain a current status. Continual progress is being made in operating systems as well as those in the planning stages. The potential of magnetic levitation is being considered and these systems may impact the future of high speed rail. While there are no high speed maglev systems in commercial use, experimental systems may provide indications that commercial use is feasible. Several high speed plans are considering utilization of the maglev approach.

Dissemination of this information to the membership is to familiarize all those interested in rail passenger transportation with a brief but concise summary, and with as much factual information as was obtainable.

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1. EXISTING HIGH SPEED RAIL SYSTEMS

1.1 Around the World

1.1.1 Japan

a. TOKAIDO (South)
From TOKYO to SHIN OSAKA
Length: 320 miles
Commissioned: October 1964
Operating speed: 137 mph since November 1986
Equipment: Shinkansen Series 0 and Series 100
Annual Ridership: 101 million in 1988
Operating Agency: Central Japan Railway Company

b. SANYO (South)
From SHIN OSAKA to HAKATA
Length: 344 Miles
Commissioned: March 1972 from SHIN OSAKA to OKAYAMA
March 1975 from OKAYAMA to HAKATA
Operating Speed: 143 mph since March 1989
Equipment: Shinkansen Series 0 and Series 100
Annual Ridership: 55 million in 1988
Operating Agency: West Japan Railway Company

c. TOHOKU (North)
From OMIYA to MORIOKA
Length: 289 miles
Commissioned: June 1982
Operating Speed: 149 mph since March 1985
Equipment: Shinkansen Series 200
Annual Ridership: 33 million in 1988
Operating Agency: East Japan Railway Company

d. JOETSU (West)
From OMIYA to NIIGATA
Length: 167 miles
Commissioned: November 1982
Operating Speed: 149 mph since 1988
Equipment: Shinkansen Series 200
Annual Ridership: 17 million in 1988
Operating Agency: East Japan Railway Company

1.1.2 France

a. TGV - SOUTHEAST
From PARIS to LYON
Length: 264 miles (length of HSR line: 259 miles)
Commissioned: September 1981 from ST. FLORENTIN to LYON
September 1983 from PARIS to ST. FLORENTIN
Operating Speed: 168 mph since September 1983
Equipment: TGV - PSE
Annual Ridership: 18.1 million in 1989
Operating Agency: S.N.C.F. (French National Railroad)

b. TGV - ATLANTIQUE (West)
From PARIS to LE MANS
Length: 125 miles (Length of HSR line: 112 miles)
Commissioned: September 1989
Operating Speed: 186 mph
Equipment: TGV - Atlantique
Annual Ridership: 19.1 million (estimation for 1992 including the Southwest branch)
Operating Agency: S.N.C.F. (French National Railroad)

1.1.3 Italy

a. DIRETTISSIMA
From ROME to FLORENCE
Length: 157 miles (130 miles are completed)
Revenue Service: Started with ETR 450 in summer 1988
Operating Speed: 155 mph
Equipment: ETR 450 tilting train
Annual Ridership: N/A
Operating Agency: FS (Italian State Railroad)

1.1.4 Miscellaneous

Several countries operate at 125 mph on upgraded conventional lines: United Kingdom, Germany, France, Italy, etc.

1.2 In North America

1.2.1 Northeast Corridor

From WASHINGTON to NEW YORK
Length: 224 miles
Project substantially complete by the end of 1984
Operating Speed: 125 mph
Equipment: AEM-7 locomotive and Amfleet cars
Annual Ridership: 5 million in 1988 (excluding commuter traffic)
Operating Agency: AMTRAK

2. HIGH SPEED RAIL SYSTEMS UNDER CONSTRUCTION

2.1 Around the World

2.1.1 Germany

a. HANOVER - WURZBURG
Length: 201 miles
Planned Operation
Start: June 1991
Operating speed: 155 mph
Equipment: ICE
Expected Ridership: N/A
Operating Agency: DB (German Federal Railroad)

b. MANNHEIM - STUTTGART
Length: 62 miles
Planned Operation
Start: June 1991
Operating Speed: 155 mph
Equipment: ICE
Expected Ridership: N/A
Operating Agency: DB (German Federal Railroad)

2.1.2 France

a. TGV - ATLANTIQUE (Southwest)
From PARIS to TOURS
Length of HSR line: 65 miles (in addition to the 77 miles of HSR line used in common with the Paris - Le Mans TGV line)
Planned Operation
Start: September 1990
Operating speed: 186 mph
Equipment: TGV - Atlantique
Expected Ridership: 19.1 million in 1992 (including the West branch)
Operating Agency: S.N.C.F. (French National Railroad)

b. TGV - NORTH
From PARIS to LILLE - BELGIUM - CHANNEL TUNNEL
Length: 207 miles
Planned Operation
Start: May 15, 1993 (with Channel Tunnel)
Operating Speed: 200 mph
Equipment: TGV - TMST (Transmanche Super Train)
Expected Ridership: 28 million in 2000
Operating Agency: S.N.C.F. (French National Railroad)

c. INTERCONNECTION OF TGV-LINES
PARIS By-pass with stop at EURODISNEY & ROISSY Intl AIRPORT
Length: 65 miles
Planned Operation
Start: Summer 1994
Operating Speed: 186 mph
Equipment: TGV - Réseau
Expected Ridership: 11 million
Operating Agency: S.N.C.F. (French National Railroad)

d. EXTENSION OF TGV - SOUTHEAST
From LYON to VALENCE
Length: 75 miles
Planned Operation
Start: 1992 from SATOLAS to ST. QUENTIN
1993 from MONTANAY to SATOLAS
1994 from ST. QUENTIN to VALENCE
Operating Speed: 186 mph
Equipment: TGV - Réseau
Expected Ridership: 2.3 million increment to TGV-Southeast
Operating agency: S.N.C.F. (French National Railroad)

2.1.3 ITALY

a. DIRETTISSIMA (Extended)
MILAN-BOLOGNA-FLORENCE-ROME-NAPLES
Length: 482 miles
Planned Operation Start: Construction has been put on hold
Operating speed: 171 mph
Equipment: ETR 500
Expected Ridership: N/A
Operating Agency: FS (Italian State Railroad)

2.1.4 JAPAN

a. HOKURIKU (West)
From TAKASAKI to KARUIZAWA
Length: 25 miles
Planned Operation Start: 1996
Operating Speed: N/A
Equipment: Shinkansen
Expected Ridership: N/A
Operating Agency: East Japan Railway Company

2.1.5 SPAIN

a. MADRID-SEVILLE
Length: 292 miles
Planned Operation Start: 1992
Operating Speed: 171 mph
Equipment: TGV-Atlantique
Expected Ridership: N/A
Operating Agency: RENFE (Spanish National Railroad)

2.2 IN NORTH AMERICA

(None)

3. HIGH SPEED RAIL SYSTEMS IN PLANNING

3.1 AROUND THE WORLD

3.1.1 FRANCE

a. TGV-EAST
From PARIS to STRASBOURG
Length: 261 miles
Operating speed: 186 mph
Equipment: TGV
Expected Ridership: N/A
Operating Agency: S.N.C.F. (French National Railroad)

b. TGV-MEDITERRANEAN
Extension of TGV-SOUTHEAST to SPAIN & FRENCH RIVIERA
Length: 404 miles
Operating Speed: 186 mph
Equipment: TGV
Expected Ridership: N/A
Operating Agency: S.N.C.F. (French National Railroad)

3.1.2 AUSTRALIA

From MELBOURNE to SYDNEY
Length: 539 miles
Operating speed: 220 mph
Equipment: Steel-wheel HSR
Expected Ridership: N/A
Operating Agency: N/A

3.1.3 BRAZIL

From SAO PAOLO to RIO DE JANEIRO
Length: 280 miles
Operating Speed: 186 miles
Equipment: Steel-wheel HSR
Expected Ridership: N/A
Operating Agency: N/A

3.1.4 KOREA

From SEOUL to PUSAN
Length: 235 miles
Operating Speed: 218 mph
Equipment: Steel-wheel HSR
Expected Ridership: N/A
Operating Agency: KNR (Korean National Railroad)

3.1.5 TAIWAN

From TAIPEI to KAOHSIUNG
Length: 210 miles
Operating Speed: 171 mph
Equipment: Steel-wheel HSR
Expected Ridership: N/A
Operating Agency: N/A

3.1.6 U.S.S.R.

LENINGRAD-MOSCOW-CRIMEA (Central-South)
Length: 1,300 miles
Operating Speed: 150-186 mph
Equipment: Steel-wheel HSR
Expected Ridership: 90 million
Operating Agency: N/A

3.2 IN NORTH AMERICA

3.2.1 CALIFORNIA-NEVADA

ANAHEIM-LAS VEGAS
Length: 270 miles
Operating Speed: 185-250 mph
Equipment: Steel-wheel HSR or maglev
Expected Ridership: 8.8-13.0 million in 2000
State Agency: California-Nevada SSGT Commission

3.2.2 CANADA

QUEBEC-MONTREAL-OTTAWA-TORONTO-WINDSOR
Length: 580 miles
Operating Speed: 186 mph
Equipment: Steel-wheel HSR
Expected Ridership: 6.6 million in 2010
Operating Agency: N/A

3.2.3 FLORIDA

MIAMI-ORLANDO-TAMPA
Length: 314 miles
Operating Speed: 150-185 mph
Equipment: Steel-wheel HSR
Expected Ridership: 1.7-5.9 million in 1999
State Agency: Florida High Speed Rail Transp. Commission

ORLANDO-MAGLEV DEMONSTRATION PROJECT
Length: 13.5 miles
Operating Speed: 250 mph
Equipment: Transrapid Maglev
Expected Ridership: 6.5 to 8.5 million in 1995
State Agency: Florida High Speed Rail Transp. Commission

3.2.4 OHIO

CLEVELAND-COLUMBUS-CINCINNATI
Length: 257 miles
Operating Speed: 125-170 mph
Equipment: Steel-wheel HSR
Expected Ridership: 3.1-4.6 million in 2000
State Agency: Ohio High Speed Rail Authority

3.2.5 PENNSYLVANIA

PHILADELPHIA-PITTSBURGH
Length: 308 miles
Operating speed: 180-250 mph
Equipment: Steel-wheel HSR or maglev
Expected Ridership: 6.6-7.5 million in 2000
State Agency: N/A

3.2.6 TEXAS

FORT WORTH/DALLAS-HOUSTON-SAN ANTONIO-AUSTIN

Length:	Dallas-Houston	247 miles
	Total Texas Triangle	618 miles
Operating Speed:	185 mph	
Equipment:	Steel-wheel HSR	
Expected Ridership:	Dallas-Houston — 2.1-4.0 million in 1998	
	Total Texas Triangle — 5.3-9.7 million in 1998	
State Agency:	Texas High Speed Rail Authority	

COMMITTEE 34—SCALES

Chairman: F. J. Loyd, Jr.

Report of Sub-Committee D-1-83

CRITERIA FOR LOCATION OF COUPLED-IN-MOTION WEIGHING SYSTEMS

By: W. G. GeMeiner (Subcommittee chairman), R. T. Brumbaugh, J. R. Caldicott, R. W. Harris, R. J. Hostetler, F. J. Loyd, Jr., N. S. Patel, D. G. Schott

Introduction

In 1975 an A.R.E.A. Special Committee on Scales published as information a set of guidelines that were designed to assist in the analysis of potential locations for coupled-in-motion scales. In the past decade, as railroads and heavy industry have automated and implemented technology to provide more efficient operations, the use of C-I-M weighing systems has increased. These scales, because of their automatic nature and the ease with which they can be integrated into other control systems, have been effectively used in coal and phosphate load-out systems. A number of railroads have them installed in main line trackage, where in many instances, entire trains are being weighed. Weighing speed has increased—there are scales that have been certified at speeds in excess of 10 MPH. This increased usage, in conjunction with longer train size and higher operating speeds, will require a more detailed examination of proposed sites than may have been necessary in the past. Subcommittee D-1-83 acknowledges the original sub-committee work as being very complete. We have revised the site selection criteria so that it may better assist engineers who are designing load-out systems, main line scales, or other integrated applications.

General

Prerequisites in selecting a site for installation of a coupled-in-motion weighing system are those which will permit the entire train to be weighed to move over the pre-determined point in such a manner that the effects upon weight transfer from car to car will be minimized during weighing functions. The train must approach the scale at a constant, steady speed and should move over the scale in a smooth, steady movement. The train should not be bunched, and train handling should be such as to prevent excessive coupler interaction. All automatic air brakes and hand brakes on cars must be fully released.

The number of cars to be handled at a given time over the weighing device should be given careful consideration. The maximum number of cars to be handled in a single train may dictate or be dictated by the grade and curvature which can be tolerated to maintain the proper weighing speed, and at the same time allow cars to move over the scale in such a manner as to produce acceptable results. Generally, the site selection should be such that the existing grade and curvature will compensate for each other so that cars will move over the device in a non-accelerating mode. Approach conditions for a train length in each direction of the scale site are more critical for a scale used for individual car weights than for a unit-train-weights-only facility, and should be considered prior to the installation of the coupled-in-motion weighing system.

To produce accurate weights, a coupled-in-motion weighing facility requires a high standard of maintenance, in fact, a standard approaching perfection of maintenance. The high standard of maintenance is necessary to assure that the approach and retreat, the ballasted track on either end of the device, and the weight or load receiving elements will be stable so that the dynamics of moving cars will be minimized.

Where possible, before a site is selected, a test train or trains having the proposed consist to be weighed should be run over the site at the proposed weighing speed(s). Observers on both the engine and on the ground should carefully check the actual train handling requirements at the proposed site.

It is desirable to prepare a questionnaire which might point out a number of items necessary to be considered for the proposed location of the weighing device. Questions contained in the evaluation should be oriented to stimulate interest in planning for operating or transportation conditions, construction, maintenance and other factors which may have an effect on the location of the weighing system.

There are seven general categories of items to be followed. These seven categories are as follows:

1. Train Handling

1.1. What will be the maximum number of cars handled in a train or cut of cars at one time?
1.2. What will be the minimum number of cars handled in a train or cut of cars at one time?
1.3. Can the train speed be reduced without use of the automatic air brake as the train approaches the scale site?
1.4. Must train be stopped before being weighed?
1.5. If stopped, can weighing speed be attained before reaching scale site without excessive coupler interaction?
1.6. Will train brakes be fully released?
1.7. Can train speed be controlled using locomotive dynamic brake?
1.8. Can train speed be controlled using locomotive independent brake?
1.9. Will train be shoved by it's rear end moving over scale?
1.10. Will train be bunched moving over scale?
1.11. Will Engineer use throttle to pull train over scale?
1.12. Will Engineer use more than throttle position No. 2 or about 200 amp power to pull train over scale?
1.13. Will Engineer control weighing speed by varying throttle position?
1.14. Will train be stretched moving over scale?
1.15. If stretched, can vertical movement of couplers be observed?
1.16. Can excessive coupler interaction be prevented?
1.17. Will train drift over the scale site?
1.18. If the train is stopped on the scale for any reason, can it be started again without excessive coupler interaction?

2. Train Operation

2.1. Will train be delayed approaching scale due to train handling characteristics?
2.2. Will following trains be delayed?
2.3. Will opposing trains be delayed?
2.4. Will scale location adversely influence train operation beyond the scale site?
2.5. Does the scale location permit holding a train to be weighed?
2.6. Does the scale location permit holding a train that has been weighed?
2.7. Must all trains pulling over the track, pull over the scale?
2.8. Will difficulty be encountered in train handling as a result of grade conditions?
2.9. Will difficulty be encountered in train handling because of over speed?
2.10. Will train stall on scale?
2.11. Is scale location suitable for bi-directional weighing?
2.12. Is scale location suitable for single direction weighing only?
2.13. Will scale location permit push and pull weighing in both directions?
2.14. Will train operation be adversely influenced due to weather conditions?

3. Physical Characteristics

(Within a train length in both directions from scale location)

3.1. Is grade satisfactory approaching scale?
3.2. Is grade satisfactory leaving scale?
3.3. Is track alignment satisfactory approaching scale?
3.4. Is track alignment satisfactory leaving the scale?
3.5. Can a train or cut of cars move over the scale without the use of sand under the locomotive?

4. Accessibility to Scale

4.1. During construction?
4.2. For daily maintenance?
4.3. For supervising operations?
4.4. For testing?
4.5. To prevent vandalism?

5. Local Operations

5.1. Will scale location interfere with local operations?
5.2. Does scale location require dead or running rail?
5.3. Will grade crossings or local ordinances influence scale operations?

6. Scale Operations & Geography

6.1. Will scale location interfere with train operations during construction period?
6.2. Will scale be influenced by high water or ground water conditions?
6.3. Will snow conditions influence operation of scale?
6.4. Can snow and ice accumulation be properly controlled?
6.5. Will scale operation be attended or unattended?
6.6. Will automatic vehicle identification be used?
6.7. Will the scale be located within Yard Limits?
6.8. Will operating, transportation or maintenance agreements enter into the selection of the scale site?
6.9. Should the weighing system fail, what course of action will be taken?
    6.9.1. Back train over the scale installation and attempt to reweigh?
    6.9.2. Hold train for scale to be repaired?
    6.9.3. Take train(s) to another location for weighing.
6.10. What type(s) of freight will be weighed on the proposed scale?
6.11. What type(s) of equipment will be weighed on the proposed scale?
6.12. Will scale be used for unit train weights?
6.13. Will scale be used for individual car weights?
6.14. Can weight data and other required information be applied to "waybill" without delay to the train?
6.15. Must a completed "waybill" document accompany the car after it has been weighed, or may it travel with another type of document?

7. Testing Requirements

7.1. Will scale location permit testing at the required intervals with minimum delay?
7.2. Is a static scale readily available for coupled-in-motion testing of scale, if it is necessary?

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